THE EFFECT OF BASE COURSE GRADATION ON THE RESULTS OF LABORATORY PUMPING TESTS

JUNE, 1957

NO. 13

by

W. P. CHAMBERLIN

Joint
Highway
Research
Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

THE EFFECT OF BASE COURSE GRADATION ON THE RESULTS OF LABORATORY PUMPING TESTS

TO: K. B. Woods, Director
Joint Highway Research Project
June 12, 1957

FROM: H. L. Michael, Assistant Director
File: 6-18-6
Project No. C36-45F

Attached is a report entitled, "The Effect of Base Course Gradation on the Results of Laboratory Pumping Tests," by W. P. Chamberlin, a member of our staff. This study was performed under the direction of Professor E. J. Yoder and was also used by Mr. Chamberlin as his thesis in partial fulfullment of the requirements for the degree of Master of Science in Civil Engineering.

The project was performed under a contract with the Corps of Engineers, New England Division and is presented to the Board as information. It is pertinent to the important subject of pumping of airfield and highway pavements.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Assistant Director
Joint Highway Research Project

HLM:hgb

Attachment

cc: D. S. Berry
A. K. Branham
J. R. Cooper
W. L. Dolch
W. H. Goetz
F. H. Green
J. T. Hallett
F. F. Havey
G. A. Hawkins
G. A. Leonards
J. F. McLaughlin
H. L. Michael
R. D. Miles
R. E. Mills
B. H. Petty
Lloyd Poindexter
C. E. Vogelgesang
J. L. Waling
K. B. Woods
E. J. Yoder

FINAL REPORT

THE EFFECT OF BASE COURSE GRADATION ON THE RESULTS OF LABORATORY PUMPING TESTS

by

W. P. Chamberlin
Research Assistant

Joint Highway Research Project
Project C 36-45F
File: 6-18-6

Purdue University
Lafayette, Indiana

June 12, 1957

Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

http://www.archive.org/details/effectofbasecour00cham

ACKNOWLEDGMENTS

The work here reported was conducted in the soils laboratory of the Joint Highway Research Project at Purdue University as a portion of a contract between the Corps of Engineers, U.S. Army and the School of Civil Engineering (Professor K. B. Woods, Head). Professor E. J. Yoder (Associate Professor of Highway Engineering, Purdue University) served as the author's major professor and directed this research.

The author expresses his sincere appreciation to Professor Yoder for his guidance and patience throughout this study, to Doctors C. W. Lovell and J. F. McLaughlin (Assistant Professors of Civil Engineering, Purdue University) for their help and suggestions, to Messrs. . B. Dean and L. E. Witek who assumed a large portion of the drafting and tedious laboratory work associated with a study of this nature and to his wife, Doris J. Chamberlin, who was responsible for the typing of the final manuscript.

In conclusion, the author is grateful to the University staff, in general, through whom he has been presented with new interests and opportunities for service and development.

TABLE OF CONTENTS

Page

ACKNOWLEDGMENTS ii

LIST OF FIGURES v

LIST OF TABLES vii

ABSTRACT viii

INTRODUCTION

  General 1
  Purpose and Scope 2

REVIEW OF GRADATIONAL REQUIREMENTS FOR RIGID PAVEMENT BASE COURSES 7

SUMMARY OF RESEARCH CONCERNING THE DESIGN OF PROTECTIVE FILTERS . 11

SUMMARY OF LABORATORY PUMPING TESTS TO DATE. 20

TEST EQUIPMENT 26

MATERIALS

  Subgrade Soil 34
    General Description 34
    Processing and Storing 37
    Preliminary Compaction Studies 37
  Base Course Materials 38
    General Description 38
    Processing and Storing 40
    Preliminary Compaction Studies 48

PROCEDURE

  Preparation of Test Samples 57
    Compacted Subgrade 57
    Base Course Material 63
    Combined Sample 68
  Tests Performed on Individual Specimens 69

Page

RESULTS 72

DISCUSSION

Introduction 78
Conditions of Testing 78
Applied Load 78
Subgrade Compaction 79
Base Course Compaction 79
Sample Preparation 85
Compacted Subgrade 85
Base Course Sample 86
Measurement of Test Results 87
Interpretation of Results 89

CONCLUSIONS 94

BIBLIOGRAPHY 97

APPENDIX 100

LIST OF FIGURES

Page

1. Limit of Stable Filter-Soil Combinations 16
2. Repeated Load Equipment 28
3. Schematic Diagram of Repetitive Loading Equipment 30
4. A Typical Time-Deformation Curve for a Subgrade Base Course Sample 33
5. Grain Size Distribution of Crosby Subgrade Soil 36
6. Moisture-Density Relationship for the Crosby Soil by the Modified AASHO Procedure 39
7. Base Course Gradations Maximum Size 3/4-inch 44
8. Base Course Gradations Maximum Size No. 4 45
9. Moisture-Density Relationships for Base Course Samples, Maximum Size 3/4-inch 50
10. Moisture-Density Relationships for Base Course Samples Maximum Size No. 4 51
11. Vibrated Density Apparatus Mounted in the Gilson Mechanical Testing Screen 53
12. Schematic Diagram of vibrated Density Apparatus 54
13. Results of Static Compaction of the Crosby Soil 58
14. Preparation of Subgrade Soil 60
15. Compaction of Subgrade Samples 62
16. Deflection Curves for Samples with Gravel Base Course 74
17. Deflection Curves for Samples with Sand Base Course 76

Page

18. Comparison of Density Values for Different Base Course Gradations 84

19. Deflection Versus Gradation for Samples with Gravel Base Course 91

20. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 0.4 100

21. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 0.5 101

22. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 0.6 102

23. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 0.8 103

24. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 1.0 104

25. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 1.2 105

26. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 1.5 106

27. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 3/4 - 2.0 107

28. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 0.5 108

29. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 0.6 109

30. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 0.8 110

31. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 1.0 111

32. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 1.2 112

33. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 1.5 113

34. Grain Size Distribution of Base Course and Pumped Soil After Test, Sample 4 - 2.0 114

LIST OF TABLES

Page

1. Summary of Classification Tests on Soils Used in This Study . 42
2. Base Course Gradations Maximum Size 3/4-inch 46
3. Base Course Gradations Maximum Size No. 4. 47
4. Minimum Density and Maximum Vibrated Density Values for Base Course Gradations in Pound per Cubic Foot 56
5. Results of Density Tests Performed on Gravel Base Course Samples 66
6. Results of Density Tests Performed on Sand Base Course Samples 67
7. Test Results for Samples with Gravel Base Course 75
8. Test Results for Samples with Sand Base Course 77
9. Tabulation of Aggregate Breakage in the Dynamic Compaction Test 82
10. Variation Between Compacted Subgrade Samples 85
11. Variation Between Compacted Base Course Samples 87

ABSTRACT

Chamberlin, William Powell, M.S.C.E., Purdue University, June, 1957. "The Effect of Base Course Gradation on the Results of Laboratory Pumping Tests". Major Porfessor: E. J. Yoder.

This thesis reports the results of a laboratory study initiated to investigate the performance of a variety of base course samples with different gradations, when these samples are placed over a standard subgrade soil and subjected to repetitions of load in such a way as to induce the pumping of fine soil to the base course surface and the intrusion of subgrade soil into the interstices between base course particles.

To accomplish the purpose of this investigation, eight gravel base course samples varying from extremes of open-graded to dense-graded and seven coarse sand base course samples, similarly graded, were placed, each at a relative density of approximately 90 per cent, over a silty-clay subgrade soil compacted to 90 per cent of maximum modified AASHO dry unit weight, and subjected to 40,000 repetitions of a 25 psi load applied at the base course surface. In each case, the load was applied through a loading head which at all times remained in contact with the base course surface. Measurements of the total deflection of the subgrade-base course system were taken intermittantly during each test. At the end of each test, the weight of soil pumped to the base course surface was measured and a grain size analysis was performed on the base course sample to determine the increase in soil finer than the No. 200

mesh sieve above the subgrade.

A description of the repetitive load equipment is included and the results of density tests performed on each sample are presented.

A good correlation was found to exist between large deflections of a subgrade-base course system and either base course pumping or subgrade intrusion. Test specimens with very open-graded gravel bases were subject to intrusion of subgrade soil and specimens with dense-graded gravel bases with an excess of 3 per cent by weight finer than the No. 200 mesh scive demonstrated pumping of fine soil sizes to the base course surface. There appeared to exist an optimum gradational range which exhibited neither base course pumping nor significant subgrade intrusion. Test specimens with coarse sand base course samples were found to perform satisfactorily over a wider range of gradation than those with the larger sized gravel bases. Test results were compared with existing filter criteria for thin base courses.

THE EFFECT OF BASE COURSE GRADATION ON THE RESULTS OF LABORATORY PUMPING TESTS

INTRODUCTION

General

In September of 1953, the Purdue Research Foundation entered into a contract with the Arctic Construction and Frost Effects Laboratory of the Corps of Engineers, U. S. Army to study base course requirements for rigid pavements constructed over frost-susceptible subgrades. The purpose of this study was to provide data to either substantiate or revise existing criteria relative to required thickness and quality in areas which experience significant frost penetration.

This investigation consisted of two phases, an extensive field and office correlative study of highway and airfield pavements (32)* and a statistical laboratory study to evaluate the relative influence of various subgrade and base course factors on the pumping of rigid pavements.(17).

The former included a literature search of current airfield and highway practices, field observations at selected air installations, and an edge sampling program and performance survey of rigid pavements in Indiana constructed with granular subgrade treatment. The latter attempted to evaluate the performance of selected subgrade-base course combinations under a repeated load situation while the factors of subgrade type and compaction, base course type and compaction and magnitude

---

*Numbers in parentheses refer to items listed in the BIBLIOGRAPHY.

of applied pressure were varied at two widely separated levels. Both phases of this investigation supported the contention that control of the gradation of base course material may provide a positive means of increasing the effectiveness of granular base courses. These researches are reviewed later in this report.

Purpose and Scope

. It was the purpose of this research program to apply the procedures developed during the aforementioned laboratory investigation to the study of the performance of a variety of base course gradations within the scope of the conditions imposed by the test equipment and in terms of certain measured variables. No attempt was made to further clarify or define the mechanics of pumping action although the author's understanding of the phenomenon is presented.

In an investigation of this nature, it is important that the event being observed be defined so that test results may be viewed with the proper perspective. No attempt is made to review the literature pertaining to pavement pumping as this has been done many times by others. The following comments, however, present the author's understanding of pumping action which may or may not be in contrast to that of others.

1. Pumping action, with respect to rigid highway or airfield pavements constructed over granular base courses, is an inclusive term relating to the movement of the finer soil sizes (sand, silt and clay) which exist beneath a pavement surface, resulting from the rather rapid movement of water carrying with it soil particles either in suspension or in motion as a result of hydrodynamic forces; the water moving under a pressure differential caused directly or indirectly by the deflection

of a loaded pavement slab. This action may be minor and produce no significant pavement distress or it may result in severe pavement damage. This action may proceed for varying periods of time before surface evidences of it are discernible. The action may be continuous, intermittant or cyclic in nature over a given period of time.

2. Pumping action may consist of any one or all of the following:

a. The movement of subgrade soil into the interstices between base course particles It has been postulated (17) that this might result from the release of pore water pressure developed in a softened subgrade soil resulting from the transmission of load through a granular base course to the subgrade surface. A distinction is made here between this aspect of pumping and the intermixing of subgrade and base course that may result from a kneading action. The latter is not considered to constitute pumping in terms of the definition given above but, rather, is related to local shearing failures of the subgrade soil at its surface of contact with the base course. The conditions considered necessary for this type of pumping to occur are: a high degree of saturation of the subgrade soil so that appreciable pore water pressures will result from small deflections of a loaded pavement, cohesive bonds in the subgrade soil which are not strong enough to resist the hydrodynamic forces of escaping pore water, a compressible subgrade soil, a base course which is many times more permeable than the subgrade, a source of available ground water and pores in the adjacent base course material of sufficient size to permit the entrance and movement of subgrade soil particles. To the author's knowledge, there is no published data which supports this conjecture. This is not to say, however, that it does not occur.

b. The movement of the finer soil sizes within and out of the base course This aspect of pumping may result from the release of pore water pressures similar to that described above or by elutriation if deflections within the base course are great enough. Conditions necessary for this type of pumping to occur are: a high degree of saturation of the base course with the presence of free water to at least the base course surface, deflection in the base course sufficient to cause the development of adequate pore water pressures, pore sizes in the base course of sufficient size to permit the movement of fine soil particles and soil particles within the base course small enough to be moved. This type of pumping has been induced in laboratory tests (17).

c. The movement of particles from and about the base course surface This aspect of pumping is erosional in nature and results from the lateral movement of free water along the base course surface as a result of displacement by a deflecting slab causing water and soil, upon occasions, to be ejected at pavement joints, cracks and edges. The conditions necessary for this type of pumping to occur are: the existance of a void space between the pavement slab and the base course surface, the presence of a small amount of free water on the base course surface and the existance of particles small enough to be moved. This type of pumping has been graphically described in the final report on the Maryland Test Road (19 p. 54).

3. The requirements for pumping action to occur are: (a) slab deflecting loads sufficient to produce soil water pressures and corresponding velocities adequate to move soil particles of varying available sizes, of which magnitude, frequency and placement of loads as well as

the load deformation characteristics of the pavement, subgrade and base course may be a measure; (b) free water, or water able to move under a pressure differential resulting from the dynamic load of a deflecting pavement slab, which water may exist at or beneath a water table, may be moving toward a water table under gravity forces or may constitute a film of perched water; and (c) soil sufficiently small to be moved either in suspension or by the hydrodynamic force of moving water and which at the same time is not held by mechanical or cohesive forces of sufficient magnitude to prevent its movement.

4. The possible effects of pumping are: (a) alteration of grain size distribution, unit weight, moisture movement and retention characteristics and strength of the components from which material has been removed or within which it has been rearranged, (b) removal of slab support by net movement of material from under the pavement, (c) cracking of pavement under wheel loads due to removal of slab support and from strength decrease of base course or subgrade, (d) cracking due to stress concentrations set up by intrusion of pumped material into a crack or joint, (e) faulting and (f) repetition of the sequence at each new crack to complete loss of structural capacity.

5. The possible visible effects of pumping are mud stains, blow holes and pumped or blown material at joints, cracks or pavement edges; and pavement cracking and faulting. Possible subsurface evidences could be cavaties beneath the slab, a concentration of fines beneath the slab, diagonal shear failures on the under side of the slab at intruded joints, and intermixing of sub-pavement soil layers of contrasting colors.

To accomplish the purposes of this investigation, eight gravel base

course samples varying from extremes of open-graded to dense-graded and seven coarse sand base course samples similarly graded were placed, each at a relative density of approximately 90 per cent, over a standard silty-clay subgrade compacted to 90 per cent of modified AASHO density, and subjected to 40,000 repetitions of a 25 psi load applied at the base course surface. In each case, the load was applied through a loading piston which at all times remained in contact with the base course surface. Thus, from this series of tests, it was not possible to investigate that aspect of pumping characterized by erosion along the base course surface. This type of test could, however, induce the movement of fines through and out of the base course as well as the intrusion of subgrade soil into the interstices between base course particles by a pumping action or by mechanical manipulation or kneading at the subgrade-base course surface of contact. It was recognized that there are other aspects of base course performance, for instance permeability, which are dependent upon gradation, but these were not considered.

REVIEW OF GRADATIONAL REQUIREMENTS FOR RIGID PAVEMENT BASE COURSES

The design of granular base courses for use under rigid pavements, from the standpoint of gradation, has for the most part, developed from the results of numerous pavement performance surveys and several highway test sections (1, 18, 19, 29). These performance surveys have been conducted and these test sections built by the highway departments of the various states and by public and private research organizations. The results of such endeavors have been general in nature, in most cases indicating trends, and have been incorporated into the specifications for base course materials of various agencies, most often in the form of (a) limits on the allowable percentage of different size fractions, (b) restrictions on the quantity of material finer than the No. 200 mesh sieve or finer than 0.02 mm, (c) a maximum allowable plasticity index of the finer soil sizes and (d) general direction toward the use of either open or dense-graded materials. Often, however, specifications have necessarily been tempered by the quality of locally available materials, the economics of altering the gradation of existing natural materials and expedients of construction. The concern of engineers regarding the gradation of granular base courses has developed primarily through attempts to control the detrimental effects that often accompany frost penetration and pavement pumping. As a result, gradational requirements vary from area to area with the factors that influence these problems (differences

in soil conditions, rainfall, temperature, traffic and available materials of construction) and the experience of the local engineers.

In September, 1954, the Corps of Engineers, U. S. Army, published a comprehensive summary of existing information on rigid pavement pumping (14). Data were obtained by means of questionnaires sent to state highway departments and Corps of Engineers offices located in areas of seasonal freezing, and supplemented by reviews of technical papers, reports on pumping and limited field observations. The information collected "in general indicated an advantage in the use of a properly graded filter course between a rigid pavement and fine-grained frost-susceptible soils." This report stated that "pumping was not reported under any conditions when a pavement was placed on a well-graded base course containing less than 10 per cent by weight of material finer than the 200 mesh sieve." A summary of answers to the pavement pumping questionnaire sent to state highway departments revealed that six of the 31 states replying reported pumping of base course materials. Of these six, two were of the dense-graded variety (more than 7 per cent passing a No. 200 mesh sieve permitted by specification), while, in three of the remaining four, pumping commenced only when the percentage of the fine grained soil in the base course was increased by entrance of the subgrade material or degradation of the base course aggregate. In addition, it was tentatively concluded that open-graded base courses are preferable where severe freezing temperatures are encountered, and, if dense-graded base courses are to be used, the binder fraction should be essentially non-plastic.

In January, 1957, Yoder (32) reported the results of an extensive study of rigid pavement pumping which included data relative to the pumping of highway pavements constructed directly on natural subgrades

and granular bases, as well as the pumping of airfield aprons, taxiways and runways. Data obtained from this study indicated that performance of rigid highway pavements built with granular bases is greatly influenced by the gradation of the base course material. Pertinent items brought out by this report are as follows:

1. For a given gradation of granular material, an increase or decrease in the per cent of material passing the No. 200 mesh sieve appears to affect the performance of the base course. This statement is supported by the observation that pavement joints associated with edge blowing were consistently found to be underlain by material containing at least three or four per cent more fines than those with no blowing. The author suggests Fuller's maximum density curve as a possible upper limit for specifications dealing with the grading of base course aggregates.

2. The permissible quantity of fines in a base course material appears to be a function of the maximum grain size of the aggregate. This statement is supported by a comparison of sands containing up to 14 per cent or more fines which show satisfactory service to gravels which prove unsatisfactory if they contain more than 10 per cent fines.

3. For comparable course aggregate bases, more fines can apparently be permitted in base courses of crushed stone than those of gravel.

Havers (17) showed that the performance of a subgrade-base course system cannot be explained solely by a consideration of the direct effects of one selected factor as base course gradation, but rather is dependent to some extent on the interaction between a number of things such as subgrade type and compaction, base course type and compaction and the magnitude of the applied load. He concluded, however, that if selection of a base course type is feasible in a given situation, one with an open

textured gradation should be chosen since it would be apt to deflect less under repeated loads than a dense-textured base until the total deflection of the latter had increased to a point where structural failure of the overlying pavement could be postulated. The report is reviewed more thoroughly in later pages.

SUMMARY OF RESEARCH CONCERNING THE DESIGN OF PROTECTIVE FILTERS

The need for a filter layer in direct contact with the subgrade soil under rigid pavements is recognized by the present Corps of Engineers' specifications (10) which read as follows:

> Where the combined thickness of pavement and base over a frost susceptible subgrade is less than the design depth of frost penetration, the following additional design requirements shall apply:
>
> (a) For both flexible and rigid pavements, the bottom 4 inches of base as a minimum shall consist of any non-frost susceptible gravel, sand, screenings, or similar material and shall be designed as a filter between the subgrade soil and overlying basecourse material to prevent mixing of the frost-susceptible subgrade with the base during and immediately following the frost-melting period. The gradation of this filter material shall be determined in accordance with criteria presented in Paragraph 2-11 of Chapter 2, Part XIII, of the Engineering Manual (11) with the added overriding limitation that the filter material shall, in no case have more than 3 percent by weight finer than 0.02 mm. Experience indicates that a non-frost-susceptible sand is particularly suitable for this filter course. On the other hand, experience shows that a fine-grained subgrade soil will work up into an improperly graded overlying gravel or crushed stone base course under the kneading action of traffic during the frost-melting period, if a filter course is not provided between the subgrade and base course.
>
> (b) For rigid pavements, the 85 percent size (the size particle for which 85% of the material by weight is finer) of filter or regular base-course material placed directly beneath pavements shall be equal to or greater than 1/4-inch in diameter. The purpose of this requirement is to prevent loss of support by pumping soil through the joints of rigid pavements.

The filter requirements, as given in reference (11) are as follows:

Backfill for Subsurface Drains

> (a) General - Placing backfill in trenches where drain pipes are located should serve a dual purpose; it must prevent the movement of particles of the soil to be drained and it must be pervious enough to allow free water to enter the pipe without clogging it

with fine particles of soil. The material selected for backfill is called 'filter material'.

(b) Theoretical design of filter material - In order to fulfill the purposes of filter material, an empirical design based upon a modification of the original method developed by K. Terzaghi has been established, substantiated by tests conducted on protective filters used for the protection of soils in the construction of earth dams.

To prevent clogging the pipe with small particles infiltrating through the openings, the following requirements must be satisfied:

$$\frac{\text{85 percent of filter material}}{\text{size of opening in pipe}} \geq 2$$

To prevent the movement of particles in the protected soil the following conditions must be satisfied:

$$\frac{\text{15 percent size of filter material}}{\text{85 percent size of protected soil}} \leq 5$$

When the protected soil is plastic and without sand or silt partings, the 15 percent size of the filter material need not be less than 1 mm.

To permit free water to reach the pipe, the filter material must be many times more pervious than the protected soil. It has been found that this condition is fulfilled when the following requirements are satisfied:

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of protected soil}} \geq 5$$

It appears that the first person to investigate the use of underdrain filters with a gradation related to that of the adjacent soil was Karl Terzaghi. Indications are that he used this technique in the early 1920's in the repair of concrete aprons of overflow dams in the Austrian Alps. The criteria established by Terzaghi for drainage filters was noted by Bertram (4) in an unpublished report on Bou-Hanifia Dam, North Africa as:

> ---- the ratio of the 15 percent size of the filter to the 15 percent size of the base* should be at least 4, and the ratio of the

---

*In literature pertaining to the design of protective filters, the term "base" is often employed to denote that soil which is being protected from erosion, while the term "filter" applies to the material or soil layer with a controlled gradation that will allow the free passage of water while preventing the intrusion of the base soil.

> 15 percent size of the filter to the 85 percent size of the base should not be greater than 4.

No mention is made of filter particle shape, whether the criteria apply to well-graded as well as uniformly-graded bases and filters, or whether these values apply equally to cohesive and non-cohesive base materials, however, the relationships apparently persisted as part of Terzaghi's thinking for he later states (31, p. 50):

> Experiments have shown that a material satisfies the essential requirements of a filter if its 15 percent size D_{15} is at least four times as large as that of the coarsest layer of soil in contact with the filter and not more than four times as large as the 85 percent size D_{85} of the finest adjoining layer of soil.

Bertram (4) in 1940 reported the results of an experimental investigation of the filter characteristics of two uniformly-graded sands by subjecting them, in various filter-base combinations, to permeability tests under hydraulic gradients of 6-8 and 13-20. The materials used were a uniformly-graded, round-grained quartz sand (Ottawa) and a uniformly-graded, angular, crushed quartz sand. The possibility of two types of failure was recognized, clogging of the pores of the filter with base material and complete passage of the base through the filter. Stability of the system (i.e. - base fines stopped moving into the filter) was indicated by the attainment of a constant permeability and conclusions were drawn from a comparison of weights and grain size distribution curves of each layer (filter and base) as placed and after a state of equilibrium had been reached. The results of these filter tests on uniform sands led to the following conclusions:

> (1) The minimum critical ratio of the 15 percent size of the filter at the limit of stability is approximately 9. The minimum critical ratio of the 15 percent size of the filter to the 85 percent size of the base at the limit of stability is approximately 6.
> (2) The critical ratios are practically independent of the

shape of the soil grains.

(3) The critical ratios are fairly constant for the range of hydraulic gradient from 6 to 20 which was investigated.

(4) The critical ratios up to which movement of soil grains is very slight are represented by sharply defined ratios of the grain size of the base material and the filter material.

(5) The above conclusions are applicable only to filters at least 50 percent compacted.

Bertram observed that his data substantiated the criteria established by Terzaghi where the use of uniform-grained materials is contemplated and that the use of the latter's criteria provides a reasonable margin of safety.

A limited number of similar tests was also performed on graded filters but no conclusions were drawn. It was suggested that it might be worthwhile to continue experimental studies with graded filters followed by a series using combinations of base and filter layers varying from well-graded mixtures to extremes of poorly graded mixtures.

In November of 1941, the U. S. Waterways Experiment Station (12) reported a series of laboratory tests performed to verfiy the results of previous investigations into the minimum grain size of filter material required to prevent infiltration of fines. In connection with this investigation, both laboratory and outdoor tests were performed to study the infiltration of filter material into various types of commercial under-drain pipes.

The filter tests were performed in permeability tubes under hydraulic gradients of 1-2 on a uniform, naturally rounded, fine sand base material and a uniform, gravelly, coarse sand filter. It was assumed that the natural soil most susceptible to piping would be a very fine sand or a coarse silt, and for this reason the above mentioned base was selected. It was further assumed that if Terzaghi's

filter criteria could be successfully applied to a filter material that would protect such a base, it could then be applied to all natural soils.

The following conclusions were reached with respect to filter materials on the basis of grain size distribution and observation:

(1) A fine material will not wash through a filter material if the 15 percent size of the filter material is less than five times as large as the 85 percent size of the fine base material.

(2) In addition to meeting the above size specifications, the grain size curves for filter and base materials should be approximately parallel in order to minimize washing of the fine base material into the filter material.

(3) Filter materials should be packed densely in order to reduce the possibility of any change in the gradation due to movement of the fines.

(4) A filter material is no more likely to fail when flow is in an upward direction than otherwise, unless a seepage pressure becomes sufficient to cause flotation or a "quick" condition of the filter.

The Providence District, U.S. Engineers, in 1942, reported the results of filter tests performed in their laboratory and in other laboratories in which the criterion for stability was based on Hazen's uniformity coefficient, C_u, for the base material and the piping ratio D_{15F}/D_{15B}. This criterion of the Providence Engineers is reproduced by Keene (23) and shown here in Figure 1. If this curve, presented by the Providence Engineers, is applied to typical soils, Keene notes that "the Providence allowable piping ratios are a little higher than 4 for uniform soils and considerably higher than 5 for well-graded soils."

The Connecticut Highway Department (23) has performed tests in 4-inch-diameter lucite tubes and in 24 x 16 x 18-inch filter boxes which gave results similar to those reported by the Providence group, and for this reason Connecticut has adopted the latter's design criteria. All tests reported by both organizations were performed using a cohesion-

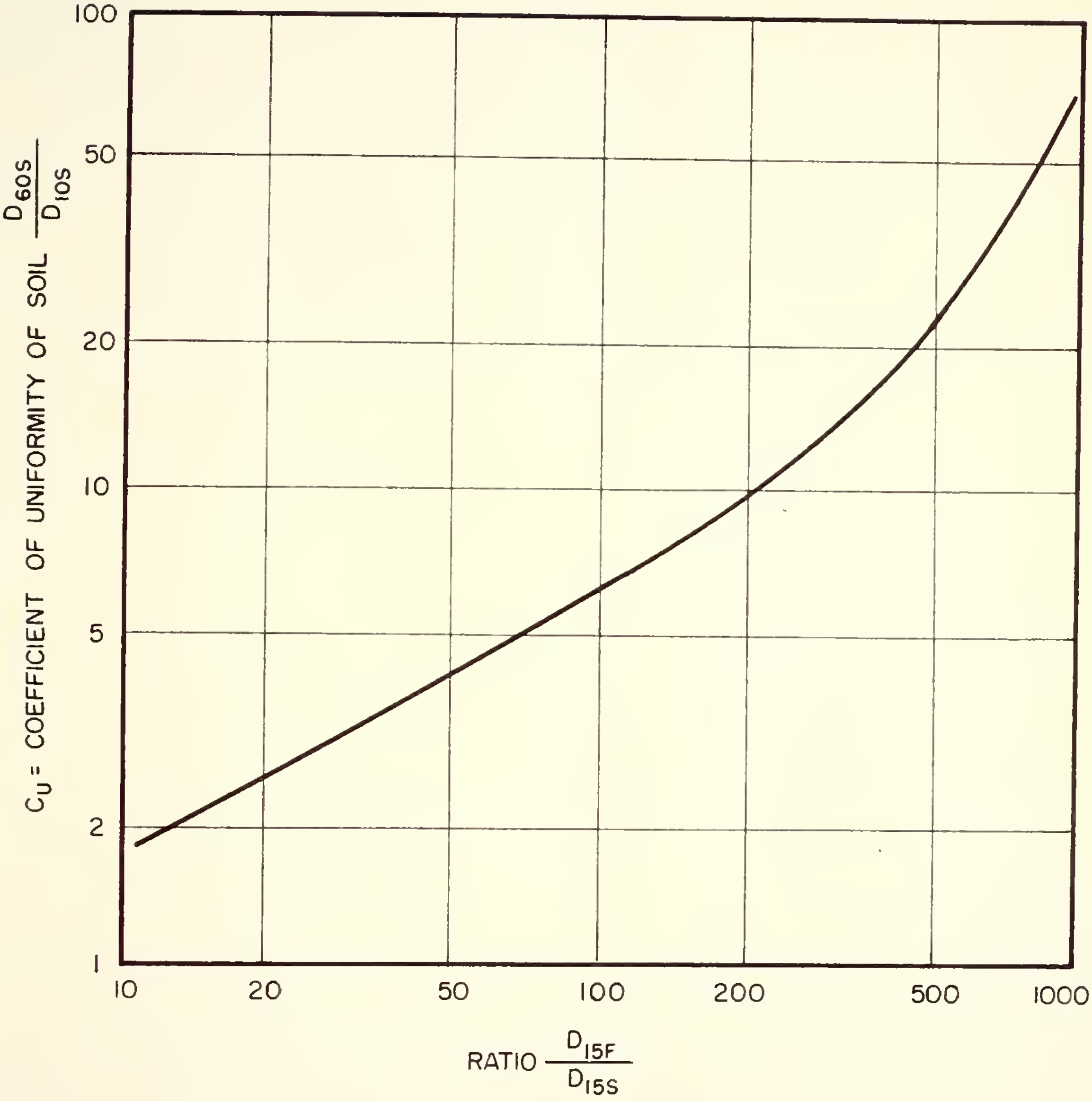


FIG. I LIMIT OF STABLE FILTER-SOIL COMBINATIONS. RATIOS TO RIGHT OF CURVE INDICATE FAILURE WILL OCCUR. TAKEN FROM PLATE NO.3, FILTER DESIGN SOILS AND PAVING LABORATORY, U. S. ENGINEER OFFICE, PROVIDENCE, R.I., 1942.

From Keene (23)

less base soil. For clay-like soils, Keene states, "piping ratios can be greater depending on the cohesion of the soil and the judgment of the engineer."

In 1955, Karpoff (22) presented the results of laboratory investigations by the Bureau of Reclamation Earth Laboratory which attempted to develop methods for designing protective filters which would be applicable to specific, severe field conditions. These tests were made in a cylindrical lucite filtration apparatus under hydraulic heads which varied from 2 to 30 feet. Relative performance of the base-filter systems was judged by observing how the quantity of flow allowed by each of the filter gradations, in combination with the same base material, varied as the hydraulic head was increased. Tests were made with uniform-grain-size and graded filter materials composed of naturally subrounded sand and gravel. The base material was, in all cases, non-cohesive. A limited number of tests was also performed on graded limestone filter materials. On the basis of these tests, the following conclusions were reached:

(1) For uniform grain-size filters consisting of natural subrounded particles - The ratio of the 50 percent grain size of filter material to the 50 percent grain size of the base material should be between the limits 5 and 10.

(2) For graded filters* consisting of naturally subrounded particles - The ratio of 50 percent grain size of filter material to the 50 percent grain size of the base material should be between the limits 12 and 58 and the ratio of the 15 percent grain size of filter material to be 15 percent grain size of base material should be between the limits 12 and 40.

(3) Tentative relationships for graded filters consisting of crushed rock - The ratio of the 50 percent grain size of filter

---

*By "graded filters" the author means "graded filter materials." In literature, the term "graded filter" usually refers to filters consisting of two or more layers of soil, each being progressively more coarse or open-graded than the adjacent layer.

material to the 50 percent grain size of the base material should be between the limits 9 and 30 and the ratio of the 15 percent grain size of filter material to the 15 percent grain size of base material should be between the limits 6 to 18.

(4) In addition to the limiting ratios established for adequate filter design —

(a) The filter material should pass the 3-in. screen for minimizing particle segregation and bridging during placement. Also, filters must not have more than 5 percent minus No. 200 particles to prevent excessive movement of fines in the filter and into drainage pipes causing clogging.

(b) The gradation curves of the filter and the base material should be approximately parallel in the range of finer sizes, because the stability and proper function of protective filters depend upon skewness of the gradation curve of the filter toward the fines, giving a support to the fines in the base material. - - -

(c) In designing of filters for base materials containing particles larger than No. 4 size the base material should be analyzed on the basis of the gradation of material smaller than No. 4 size.

It appears that the problem of protecting cohesionless soils with either uniform or well-graded filters of rounded or angular particles from the effects of seepage under constant hydraulic heads has been thoroughly investigated and that these investigations have led to adequate design criteria. It has been recognized that the soils most difficult to protect are the fine sands and the cohesionless silts. Probably, for this reason, little attempt has been made to investigate filter characteristics required to protect clay-like soils other than to recognize that they are less susceptible to the disrupting effects of seepage forces due to cohesive bonds which exist between individual particles. The philosophy seems to be that a filter material that will protect the soil believed to be the least stable, will protect all other soils.

In addition, there has been general agreement among investigators that (a) an increase in density or degree of compaction aids filter

performance (4 and12), (b) proper functioning is dependent to a large degree on the fine soil sizes (minus No. 4) and for this reason the grain size distribution curve of the filter material should be approximately parallel to that of the soil being protected in the range of the finer sizes (12 and 22), and (c) a filter material is no more likely to fail when flow is reversed than otherwise (4 and 12).

For his tests on uniformly-graded sands, Bertram (4) states that "The critical ratios are practically independent of the shape of the soil grains." Karpoff's (22) results with graded filter materials, on the other hand, indicate that crushed rock may require lower critical ratios between the D_{50} sizes and the D_{15} sizes than for naturally rounded and subrounded sands and gravels. This latter situation is probably due to the fact that the effect of angularity in most aggregates is to materially decrease density in both the loose and dense conditions (6).

SUMMARY OF LABORATORY PUMPING TESTS TO DATE

In June of 1956, John A. Havers (17) submitted a thesis to the faculty of Purdue University titled "A Study of the Interactions of Selected Combinations of Subgrade and Base Course Subjected to Repetitive Loading Tests." This thesis reported the results of a laboratory study conducted at Purdue University under contract to the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers, U.S. Army. The study was initiated to investigate, under controlled laboratory conditions, the effects of a repetitive-type of loading on selected combinations of base course material and subgrade soil, with particular reference to the problems of rigid pavement pumping. The relative influence of subgrade and base course types on a set of measured variables was determined by the statistical procedures of analysis of variance.

The laboratory study undertaken by Mr. Havers was designed as a complete factorial experiment to investigate at two levels the factors of subgrade type, base course type, subgrade compaction, base course compaction and applied pressure. Subgrade samples were molded from two soils, a plastic limestone residual clay from southern Indiana and a moderately plastic clay from the vicinity of Lafayette, Indiana. Two gradations of base course were also studied, both of which were well-graded mixtures artificially prepared by recombining in fixed proportions the various size fractions of a glacial terrace gravel with

a maximum size of 3/4-inch. The first of these base course mixtures, termed dense-graded, contained 7 per cent by weight passing the No. 200 mesh U.S. sieve while the second, termed open-graded, contained no material smaller than the No. 80 mesh U.S. sieve. Two levels of subgrade compaction were investigated, 95 per cent of the maximum unit weights obtained in the standard and modified AASHO compaction test procedures. Likewise, base course samples were placed at relative densities of 0.75 and 0.95. Each combination of the above factors was subjected to repetitions of both a 10 psi and a 40 psi pressure applied at the base course surface.

For this study it was postulated that pavement pumping might result from two separate processes.

> For the first of these to be operative, it is necessary that small accumulations of free water be present directly under a rigid pavement slab. Such a condition might conceivably exist during a frost melting period or after prolonged rainfall, since perfect contact between slab and base course is generally not maintained at all points and at all times. Deflection of the loaded pavement slab will cause this water to flow laterally over the surface of the base course to outlets at cracks, joints and pavement edges. Soil particles may be carried along with the water, and a mixture of water and soil ejected from beneath the pavement. Such movement of soil is erosional in nature, and will tend to become progressive. The size of soil grains which may be removed by this process will be governed by the pavement deflection, the clear space between the pavement and the base course, and the size of the openings through which the particles may escape.
>
> A second type of pumping could conceivably result from the movement of fine soil particles into or through the base course, actuated by pore water pressures within the subgrade and by a kneading action at the contact surface between the subgrade and base course. To create this condition, it is necessary that the soil have a high degree of saturation so that appreciable pore water pressures will result from small deflections of the loaded pavement. It is further necessary that the base course immediately above the pavement [should read subgrade] shall have openings of sufficient size to permit the entrance and movement of the finer soil particles. Removal of the finer particles of a densely-graded base course by similar means is also a possibility, and the same criteria apply for its occurance.

To simulate the conditions under which the two postulated processes of pavement pumping might occur, two types of tests were designed. In both types, a 4-inch-thick base course was placed on top of a 4-inch-thick subgrade compacted in a 7-inch inside diameter lucite cylinder. Both subgrade and base course were saturated and the combination loaded repeatedly at a controlled interval and duration by a piston at the base course surface. In one type of test, termed "single-acting," the loading piston was allowed to remain in contact with the base course surface at all times, while in the "double-acting" tests the piston returned to a zero position after each stroke such that the length of stroke increased as the cumulative deflection of the base course-subgrade system increased.

This program required the performance of 64 individual repetitive loading tests on the various combinations of base course material and subgrade soil. In addition, a certain number of tests were performed directly on the subgrades alone and on the base courses alone.

In order to compact subgrade samples to the desired density level and maintain a high degree of uniformity from sample to sample and within individual samples, a procedure of static compaction was adopted. Curves of static compactive pressure versus optimum molded moisture content and static compactive pressure versus maximum dry unit weight were developed for each of the two subgrade soils. These curves provided the basis for static compaction of subgrade samples.

Base course samples were placed to the desired density ratios*

---

*Density ratio is defined by Lane (25) as $D_r = \frac{d_n - d_o}{d_{100} - d_o}$. These symbols are defined later under PROCEDURES.

on top of the subgrade by a combination of dynamic compaction and vibration of samples which had been brought to a moisture content slightly in excess of optimum. Minimum densities were obtained by pouring the oven-dry material from a scoop into a 4 1/2-inch by 6-inch diameter mold from a height which at no time exceeded two inches. The tared mold and contents were then weighed to the nearest 0.01 pound and the dry unit weight computed. The maximum dry unit weights obtainable by the modified AASHO compaction test (performed according to Corps of Engineers' Specification 8C7.1, reference 13) were taken as the maximum attainable densities. Comparison of these maximum density values with those of the Providence vibrated density procedure showed a difference of 6.5 pcf for the open-graded base and 2.1 pcf for the dense-graded, the Providence values being the greater in both cases. These differences were attributed to the surcharge and greater intensity of vibration of the Providence test.

Each base course-subgrade combination was saturated from the bottom of the subgrade under a static pressure of approximately 2-3/4 psi. The system was then closed to prevent water from entering or leaving. A maximum of 40,000 repetitions of load was applied to each combination through a loading piston actuated by compressed air and electrically controlled for loading interval and duration.

The variables measured for each test were: total weight of material pumped to the top of the base, cumulative deflection of the loading piston, and the increase in weight of the minus No. 200 size fraction within the base. In addition, visual observations of the performance of subgrade and base course were recorded during and after the

testing of each sample.

Of the three variables mentioned above, the measurement of the increase in weight of the minus No. 200 fraction of the base course was believed to be the least accurate because of the human factor involved in separating the base course from the subgrade soil in order that a mechanical analysis might be made. It was felt that the other variables could be measured with the assurance of relatively high precision.

It was believed by the author that very satisfactory control was maintained over the factors of subgrade type, base course type, subgrade compaction and applied pressure. The static compaction procedure employed for the subgrades permitted accurate reproduction of unit weights with variation in weight between specimens generally not exceeding 1 per cent of the wet unit weight. Compaction of the base course was considered to be, by far, the least reproducible of the factors studied due to stratification and variations in density which undoubtedly existed within the samples, and due also to the impracticability, in some cases, of compacting the base to exactly the desired height.

The following conclusions were drawn from the results of the repetitive loading tests:

> 1. Although the effect of subgrade type was not pronounced, samples with Frederick subgrades were, for the same conditions of test slightly superior to those with Crosby subgrades. After the full number of loads had been applied, the specimens with Crosby subgrades had generally deflected more, pumped more material to the surface of the base course, and moved more P. #200 [minus No. 200 material] from the subgrade.
> 2. Samples with open-textured base courses deflected less under repeated loads than those with dense-textured bases until the total deflections had increased to a point where structural failure of an overlying pavement could be postulated. Intermixing of the subgrade and granular material was most extensive with

the open-graded bases, as shown by their greater P. #200 [minus No. 200] increase and by visual observation of the samples after testing. Specimens with the less-permeable base, however, had more P. #200 [minus No. 200] material moved from the subgrade and more material pumped to the surface of the base course. If selection of base-course type is feasible in a given situation, it is therefore recommended that one with an open-textured gradation be chosen.

3. Since the open gradation proved superior to the less-permeable one despite the fact that the latter satisfied several accepted criteria for the design of filters (4) (14) (25) [(4) (31) (12)] and the open one did not, it appears possible that these criteria are not fully adequate for the design of a filter to which repetitive loading is applied, when the filter is placed over a fine-grained subgrade. This is recognized in part by the current Corps of Engineers' filter requirements (21) [(11)] which state that the D_{15} size of a filter placed over a subgrade which is plastic and contains no sand or silt partings need not be less than one millimeter.

4. Increasing the compaction of the subgrade in every case decreased the deflections, decreased the weight of material pumped, and the amount of P. #200 [minus No. 200] material moved from the subgrade. If this improvement can be shown to be permanent, a high degree of compaction of the subgrade should afford an inexpensive and generally applicable method of improving the performance of rigid pavements placed on fine-grained subgrades.

5. In order for pumping to occur in these tests, it was found necessary that the water level be raised at least to the bottom of the loading piston so as to form a closed system with the base course and subgrade. As applied to a pavement, this would suggest that removal of subgrade material by pumping can occur only at those periods when the water table is at or above the surface of the base course.

6. The magnitude of the applied pressure was found to have a major effect on the variables studied, with the larger pressure tending to produce greater deflections, more extensive intermixing, and an accelerated pumping of both subgrade and base course as compared with smaller pressure. It is indicated, therefore, that within the obvious economic limitations it should be beneficial to reduce the contact pressure between the pavement and the base course by increasing the thickness of the pavement slab.

7. Increasing the number of load applications increased the total deflection of the subgrade-base course combination for all samples tested.

8. Large deflections and a large applied pressure promoted degradation of the base course during the tests. The extensive degradation which apparently occurred for some of the samples tested, however, is not considered a likely occurrence beneath pavements in service.

TEST EQUIPMENT

The loading device used for testing subgrade-base course combinations was that which was designed and built by Havers (17). Essentially this piece of equipment consists of a piston mounted vertically in a loading frame and activated by compressed air. The pressure applied at the piston face can be controlled for magnitude by a regulator in the air line and for duration and interval by a set of inlet and exhaust valves activated electrically by a timing device.

Havers (17) described the component parts of the repeated load equipment and their function as follows:

> Compressed air at 100 to 110 psi outlet pressure was supplied by a Gardner-Denver model 329 LT air compressor, powered by a 7-1/2 hp. electric motor. From the compressor tank a one-inch diameter galvanized pipe led through an air line filter to remove moisture and any foreign particles from the air, and thence into a Grove GB 307-06 Powreactor Dome pressure regulator. This regulator, of the diaphragm type, was found to regulate the outlet pressure quite accurately while still permitting the sudden and intermittent flow of air required to produce the repetitive type of loading.
>
> From the pressure regulator the line passed through an air line lubricator and branched to enter two Bellows 455V 3/4 Bel-Air electrically-controlled valves, each of which was connected to one end of a Bellows 8002 FR Power Dome air cylinder. This cylinder was mounted vertically to the upper platen of a loading frame and afforded a downward stroke of from zero to two inches for the loading piston. A loading head attached to the loading piston applied load directly to samples of subgrade and base course placed on the lower platen beneath the air cylinder. The total load which was applied could be controlled by adjusting the Powreactor pressure regulator, while both the duration of load and the time interval between successive load applications could be regulated through the Bel Air valves by means of an Agastat DED-11 double acting timer. Any desired number of loading cycles between zero and 99,999 could be preset by means of a Durant model SP predetermined counter. Manual valves and switches were so arranged that one Bel-Air valve

> could be removed from the system if desired, resulting in a single-acting stroke of the loading piston rather than its normal reciprocating action.
>
> The Grove regulator could be adjusted to approximately the desired pressure for any test series by observing a dial guage connected directly to the pressure dome of the regulator - - - -

For the series of tests here reported, adjustments were made so that only the single-acting type of movement of the loading piston was used. Thus, valves 1, 5 and 8 (Figure 3) remained open at all times while valve 7 was closed, removing one of the Bel-Air valves from the system.

The arrangement used to measure the deflections of a subgrade-base course system utilized a linear variable differential transformer (L.V.D.T.) with suitable recording apparatus. This set up is described by McLaughlin (27) as follows:

> - - - - movement of the loading piston was measured through a Brush model B6-320 Universal Analyzer and recorded directly as a deflection-time curve on a Brush model BL-202 Oscillograph. The core of the transformer was mounted on the loading piston while the coil was mounted on an arm fixed to the loading frame. As the cumulative permanent deflection in any given test might be expected to exceed 0.04-inches, the range of the L.V.D.T. in one direction, the core of the transformer was actually mounted on the end of a brass rod the other end of which rested on the end of a micrometer-screw. This micrometer-screw in turn was attached to the loading piston so that actually any vertical movement of the loading piston resulted in the same vertical movement of the core of the transformer. This arrangement enabled one to extend the useful range of the transformer because when the permanent deflections had accumulated to the point where the core was near its extreme limit, the micrometer-screw could be turned so that the core was moved back up into the coil. Readings on the micrometer-screw told how much the core had to be reset at any given time and this factor was simply added to all further deflections measured on the oscillograph.- - - -

The only modification to that described above was the use of an additional micrometer to extend the usefulness of the transformer to the full 2-inch throw of the loading piston, in anticipation of deflections in excess of one inch. This, however, was not needed.

The repeated load test equipment is shown in Figures 2 and 3. The

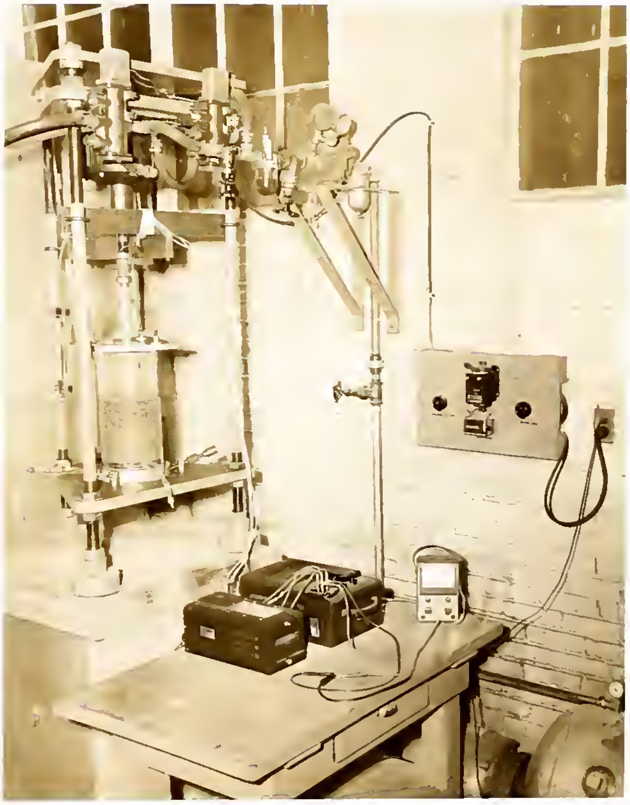


FIG. 2 REPEATED LOAD EQUIPMENT

numbered items of Figure 3 are listed on page 31. A typical time versus deformation curve for the oscillogram recorder is shown in Figure 4.

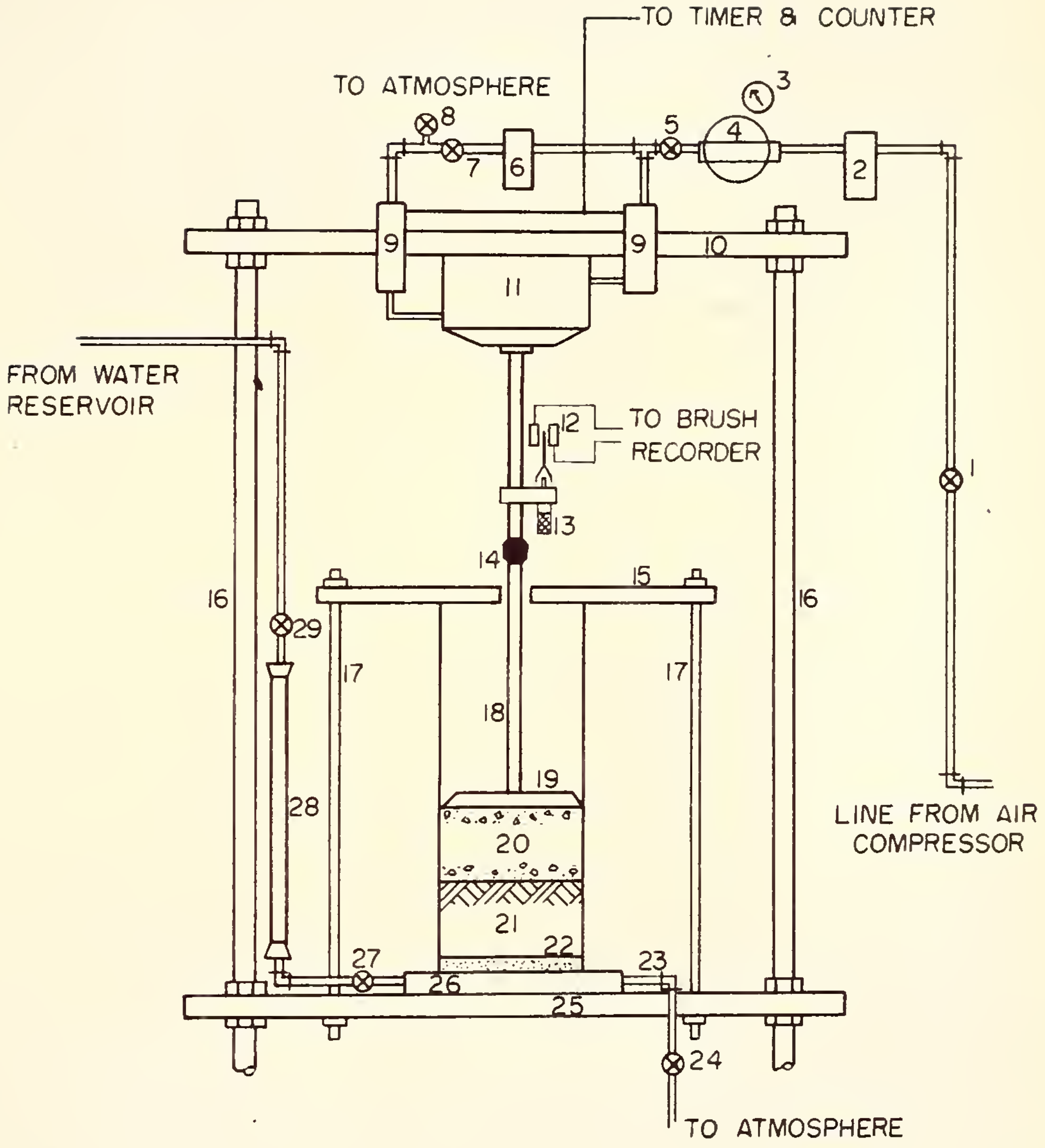


FIG. 3 SCHEMATIC DIAGRAM OF REPETITIVE LOADING EQUIPMENT

KEY TO FIGURE 3

Number	Description of Part
1.	Air line gate valve, one-inch Lunkenheimer
2.	Air line filter, 3/4-inch
3.	Pressure gauge, Crosby, 0-60 pounds
4.	Grove GB 307-06 Powreactor Dome pressure regulator
5.	Gate valve, 1 1/4-inch Lunkenheimer
6.	Air line lubricator, 3/4-inch
7.	Gate valve, 3/4-inch Lunkenheimer
8.	Gate valve, 1/4-inch Lunkenheimer
9.	Bel-Air 4SSV 3/4 valves
10.	Upper platen of testing machine
11.	Bellows 8002 FR Power Dome air cylinder
12.	L.V.D.T. assembly
13.	Micrometer-screw
14.	Coupling device
15.	Aluminum cover plate, 3/4-inch thick
16.	Guide rods, 1 1/2-inch diameter, for platens of testing machine
17.	Tie rods, 3/8-inch diameter
18.	Extension rod for loading head, one-inch diameter
19.	Loading head, 6.97-inch diameter and one-inch thickness, with bevelled edge
20.	Compacted base course sample
21.	Compacted subgrade sample
22.	Porous stone, 7 1/16-inch diameter and one-inch thickness

Number	Description of Part
23.	Water line, 1/4-inch Saran plastic tubing
24.	Needle valve, 1/4-inch
25.	Lower platen of testing machine
26.	Aluminum base plate, recessed for lucite cylinder
27.	Needle valve, 1/4-inch
28.	Standpipe
29.	Needle valve, 1/4-inch

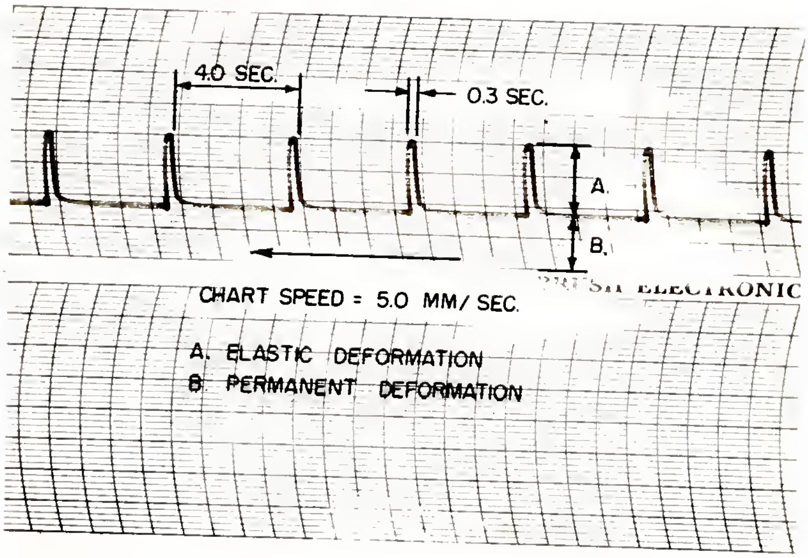


FIG. 4 A TYPICAL TIME-DEFORMATION CURVE FOR A SUBGRADE-BASE COURSE SAMPLE

MATERIALS

Subgrade Soil

General Description

The soil chosen for the molding of subgrade samples was a light-brown silty clay collected in the vicinity of Lafayette, Indiana. This soil was taken from the "B" horizon of the Crosby profile which begins at a depth of about four inches below the ground surface and extends to a depth which may reach fifty inches (3). The Crosby profile occupies a considerable portion of the gently undulating Tipton Till Plain of the Older Wisconsin Drift (3). As such, it provides the subgrade support for many miles of highway pavement in this area.

In Havers' study (17), two soils were used, a highly plastic limestone residual clay from southern Indiana, termed Frederick, and the Crosby soil described above. With respect to the test series he performed in the laboratory, Havers states, "- - - - combinations with the more plastic Frederick subgrades were found to perform slightly better than those with the less plastic Crosby subgrades."

The Atterberg limits of the Crosby soil performed on random samples of the air-dried processed soil are as follows:

Liquid Limit = 37.2 %

Plastic Limit = 19.4 %

Plasticity Index = 17.8 %

The specific gravity of solids was found to be 2.69.

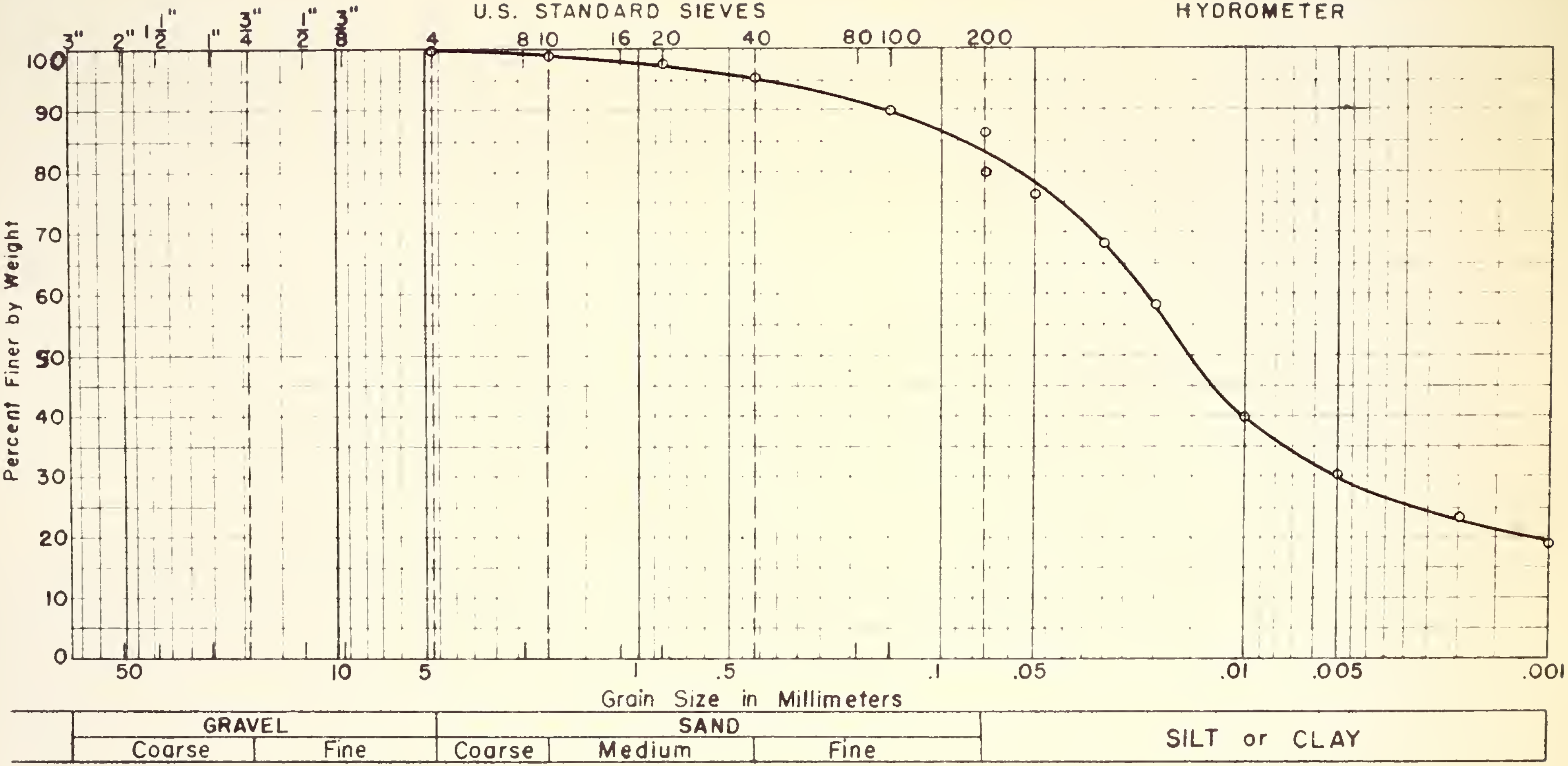
In Figure 5, is represented the grain size distribution. Imposed beneath this gradation curve are the textural ranges of the Corps of Engineers' unified soil classification system (15). The Crosby soil is a fine-grained,* non-uniformly graded, moderately plastic soil with small amounts of sand.

The values given above by the various engineering classification tests are not necessarily typical of all soils named as Crosby, nor do they necessarily represent the characteristics of the Crosby soil in its natural state. Reference 3 (p. 349), gives values of the liquid limit and plasticity index for soils classified as Crosby collected in different parts of Indiana with ranges of 21.0 per cent to 51.4 per cent and 6.0 per cent to 35.0 per cent, respectively. Thus, the Atterberg limits, specific gravity, and grain size distribution may vary from deposit to deposit and may vary with depth within any given deposit. Further, these tests were performed on the soil in an air dried condition. It has been established that certain irreversible changes take place in a clay-like soil when its moisture content has been reduced to a value below the shrinkage limit of that soil. In consideration of these comments, the soil used in this investigation should not be thought of as typical of any particular soil or soil group but only as falling within the broad catagory of a silty or moderately plastic clay.

All of the above classification tests were performed in general

---

*The term fine-grained, as such, is descriptive only in a qualitative sense. The unified soil classification system (15) however, specifies that in order for a soil to be fine grained, a minimum of 50 per cent by weight of the soil must pass through the No. 200 mesh sieve.



Remarks

GRAIN SIZE DISTRIBUTION
OF CROSBY SUBGRADE SOIL
FIG. 5

From Havers (17)

accordance with procedures outlined in reference (24).

Processing and Storing

The Crosby soil, soon after excavation, was forcibly seived through 1/4-inch hardware cloth in the field. After seiving, the soil was brought into the laboratory, air dried, thoroughly mixed by shoveling on a concrete floor and stored.

In order to facilitate uniform moisture distribution, the partially processed soil was taken from storage and pulverized to a powder-like texture in a Los Angeles abrasion machine with a charge of 20 standard steel spheres. The pulverized soil was then mixed by shoveling on a concrete floor and random samples were taken to determine the adequacy of mixing. Atterberg limits performed on these random samples gave a range in liquid limit of 0.9 per cent moisture and a range in plastic limit of 1.0 per cent moisture. It was believed that these discrepancies were similar in magnitude to those which would be expected had successive tests been performed on the same sample; and, for this reason, that the results of these tests indicated adequate mixing of the pulverized soil.

The completely processed soil was then stored in 10-gallon metal containers for future use.

Preliminary Compaction Studies

The moisture-density* relationship was determined for the Crosby soil in general accordance with AASHO designation T99-49 (2) which specifies compaction in a standard 4.6 x 4-inch-diameter Proctor mold.

In order to establish each point on the moisture-density curve, a

---

*More properly, dry unit weight.

predetermined quantity of distilled water was added to five pounds of the pulverized Crosby soil. This distilled water was incorporated by means of an electric paint sprayer and continuous troweling of the soil. When the desired batch weight was attained, as determined by weighing to the nearest one-hundredth of a pound, the galvanized pan containing the moist soil was covered with a plastic cloth and stored for a minimum of 24 hours in a constant humidity room in order that the added moisture might become more evenly distributed through the soil. After curing, the soil was compacted in three layers at 25 blows per layer with a 10-pound hammer falling freely through a vertical distance of 18 inches (2). After making the proper weighings and taking samples for moisture determination, the compacted soil was then discarded and a new batch prepared at a different moisture content.

The maximum dry unit weight attained following this procedure was accordingly referred to as 100 per cent modified AASHO density or compaction, for the Crosby soil. The moisture-density relationship for the Crosby soil is shown in Figure 6.

Base Course Materials

General Description

The material selected for use in preparing base course samples was washed glacial terrace gravel obtained commercially from a local source. This material was chosen because it was readily available and is of a type commonly used for subgrade treatment under rigid pavements in northern Indiana (21). A dune sand supplied the size fraction contained between

TEST DATA						
w	3.6	7.1	12.6	13.6	15.7	16.4
γ_o	102.2	110.6	118.1	118.8	114.8	113.2

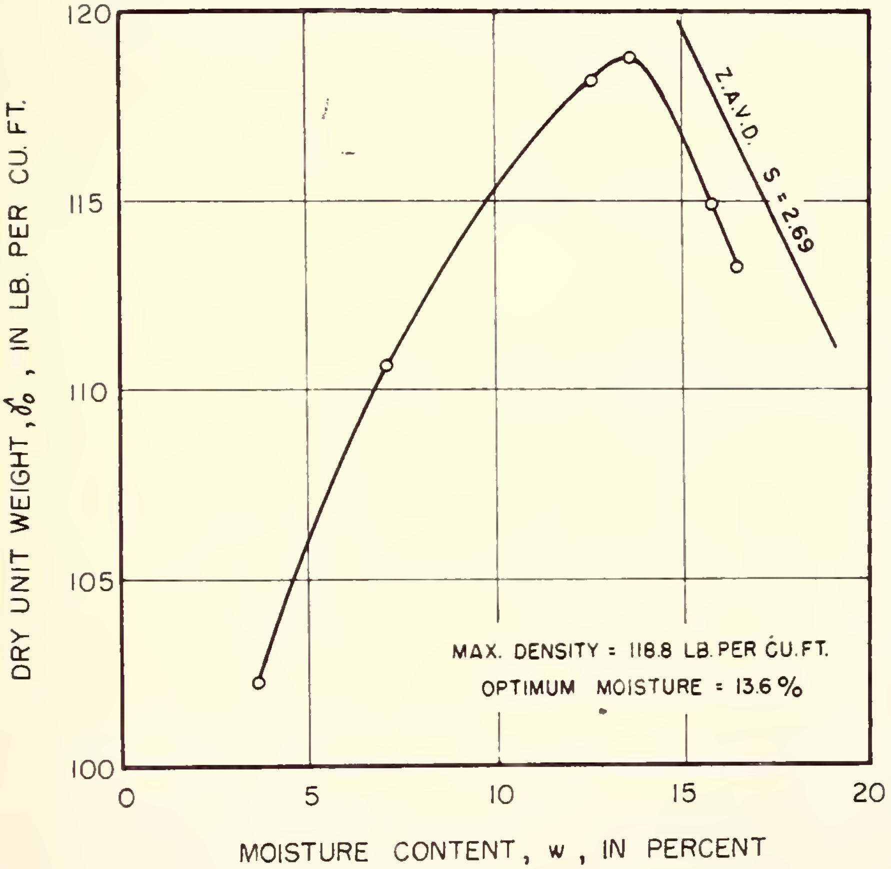


FIG. 6 MOISTURE-DENSITY RELATIONSHIP FOR THE CROSBY SOIL BY THE MODIFIED AASHO PROCEDURE

the No. 80 and No. 200 U. S. Bureau of Standards sieves and a silt of low plasticity, pedologically classed as Vigo, was used as a minus No. 200 filler. The particle shape of the individual gravel pieces ranged from subrounded to well-rounded.

Processing and Storing

Washed gravel was obtained and brought into the laboratory where it was mechanically separated into the sizes retained between the following U. S. Bureau of Standard seives: 3/4-inch, 1/2-inch, 3/8-inch, No. 4, No. 10, No. 20, No. 40 and No. 80. All material finer than the No. 80 as well as all material coarser than the 3/4-inch sieve was discarded.

The maximum size of aggregate was abritrarily selected as 3/4-inch to conform to the dimensions of the testing cylinder. A study of construction records (21) and the results of rigid pavement edge sampling (32) indicate that this size is essentially the same as the maximum aggregate size used in Indiana for rigid pavement subgrade treatment.

After separation into the above sizes, the gravel was washed, oven dried and stored in clean canvas bags until used.

Dune sand, predominatly quartz, was brought into the laboratory, oven dried and sieved. During the sieving operation, root hairs and cohesive soil lumps were removed. The portion retained between the No. 80 and No. 200 sieves was washed, dried and stored in clean canvas bags. The portions retained on the No. 80 sieve and passing the No. 200 sieve were discarded.

A quantity of Vigo silt was available in the laboratory from previous studies. This soil was in an air-dried condition and had been

screened to the extent that no lumps in excess of 1/4-inch existed. The soil, in this state, was pulverized to a fine powder as was the Crosby soil. The silt was then sieved through a No. 200 U. S. Bureau of Standard's sieve and the coarser fraction discarded. The resulting minus No. 200 pulverized silt was then stored in 10-gallon metal containers for future use.

Atterberg limits performed on the processed Vigo silt gave the following results:

Liquid Limit = 33.9 %

Plastic Limit = 24.0 %

Plasticity Index = 9.9 %

Artificial Gradations

The size gradation of aggregate used in subgrade treatment is recognized as one of the more important factors influencing the performance of rigid pavement base courses and, in fact, it is this single factor of gradation which has received most of the recognition from design criteria and specifications pertaining to base course and protective filter design (4, 31, 12, 20 and 22).

Implied in the concept of gradation are curve shape, degree of fineness and particle size range (8). These, as well as particle shape, have an important influence on the densities that can be attained with granular materials (6). For this study, only one curve shape was considered, that which is essentially concave. It was recognized that other distributions were possible, some quite common, but it was felt that this shape best represented the pit or bank-run sands and gravels which are widely used in Indiana for subgrade treatment.

ATTERBERG LIMITS

SOIL	PL (%)	LL(%)	PI (%)
CROSBY SILTY CLAY	19.4	37.2	17.8
VIGO SILT	24.0	33.9	9.9

GRAIN SIZE DISTRIBUTION

SOIL	PERCENTAGE FINER THAN			
	NO. 4	NO. 10	NO. 40	NO. 200
CROSBY SILTY CLAY	100	99	96	84
VIGO SILT	100	100	100	100

MODIFIED AASHO MOISTURE-DENSITY RELATIONSHIP

SOIL	MAXIMUM DRY UNIT WEIGHT (PCF)	OPTIMUM MOISTURE CONTENT (%)
CROSBY SILTY CLAY	112.8	13.6
VIGO SILT	NOT PERFORMED	

SPECIFIC GRAVITY OF SOLIDS

CROSBY SILTY CLAY	2.69
VIGO SILT	2.70

TABLE I SUMMARY OF CLASSIFICATION TESTS ON SOILS USED IN THIS STUDY

Two maximum sizes were selected for study, 3/4-inch and No. 4. A set of distribution curves was developed for each maximum aggregate size, based on Talbot and Richart's (30) mathematical expression:

$$p = 100(\frac{d}{D})^n$$

where "p" is the per cent of material by weight which passes a given sieve having openings of width "d", "D" is the maximum particle size of the given aggregate, and "n" is a variable exponent. If "n" equals 1.0, the curve on a linear plot will be a straight line. If "n" equals 0.5, the curve will be a parabola which represents the ideal grading curve for maximum density shown by Fuller and Thompson (16).

For each of the two maximum sizes, a family of curves was obtained by varying the exponent "n". The gradation curves selected for study were chosen so as to be equally spaced with respect to one another and so as to cover the region between the natural limitation of uniformity and the practical limitation of minus No. 200 material. These curves are shown in Figures 7 and 8.

Each base course gradation was designated by a code number consisting of two parts, the maximum particle size and the "n" value of the gradation curve, such that a sample designated as 3/4 - 0.8 would have a maximum particle size of 3/4-inches and be graded according to the formula $p = 100(\frac{d}{D})^{0.8}$. The relative percentages of the different size fractions are shown for each base course sample in Tables 2 and 3.

Only three of these gradations (3/4 - 0.4, 4 - 0.5 and 4 - 0.6) meet the design criteria first proposed by Terzaghi (31) and later modified and adopted by the Corps of Engineers, U.S. Army (12 and 10) for granular subgrade treatment. These same three, however, plus sample

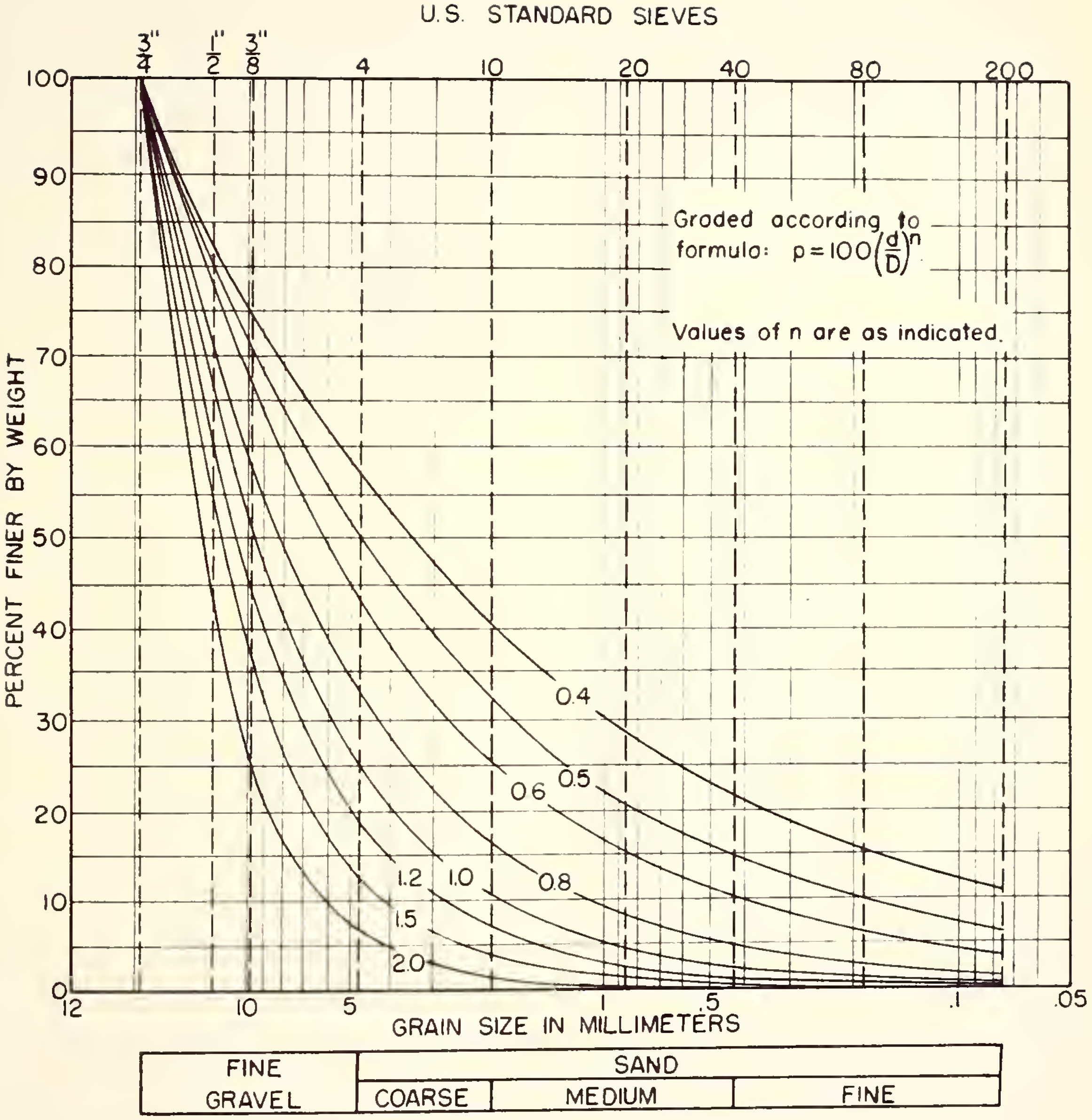


FIG. 7 BASE COURSE GRADATIONS
MAXIMUM SIZE $\frac{3}{4}$-IN.

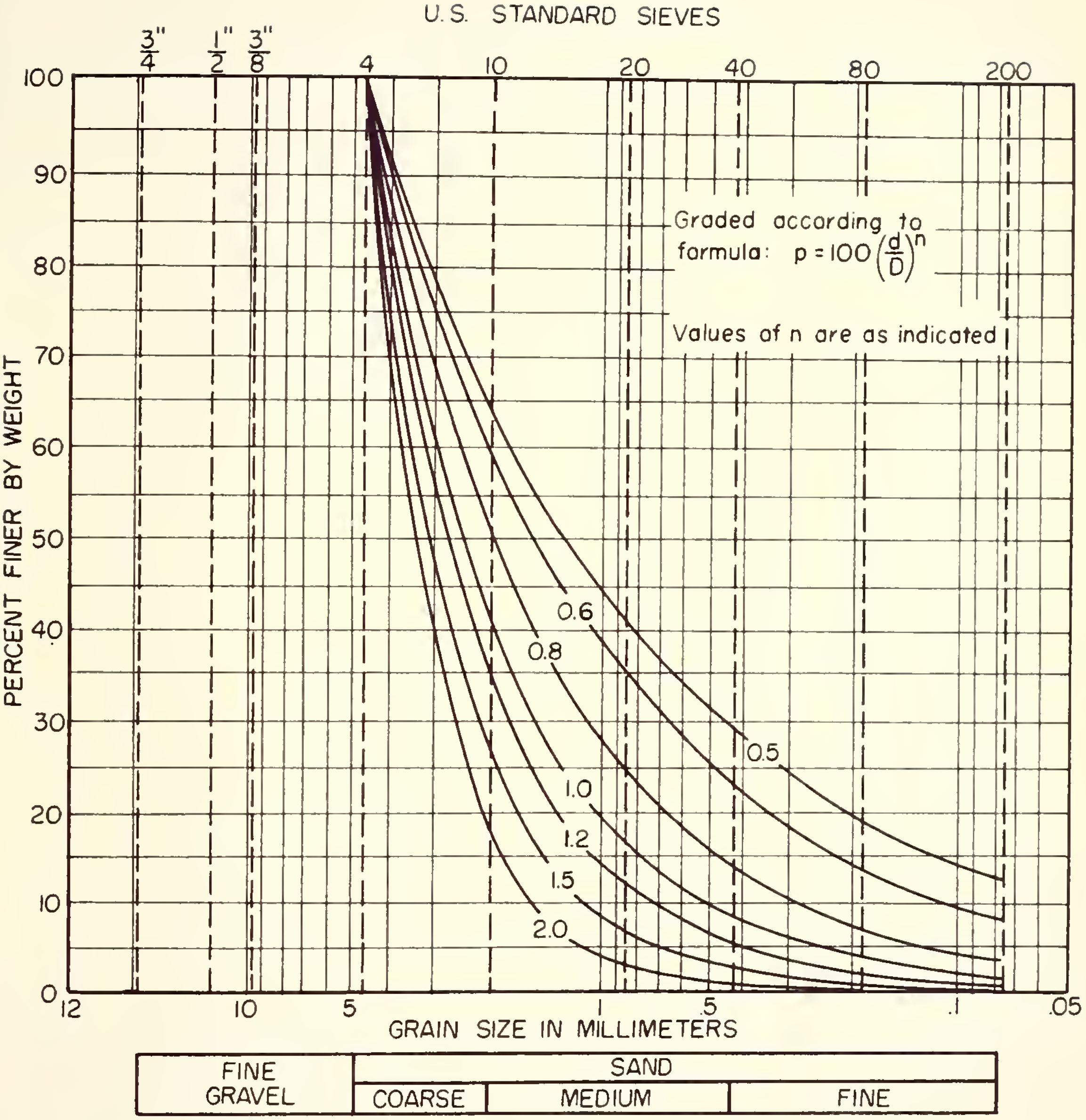


FIG. 8 BASE COURSE GRADATIONS
MAXIMUM SIZE NO. 4

SAMPLE	PERCENTAGE RETAINED BETWEEN				
	FINE GRAVEL	SAND			SILT OR CLAY
		COARSE	MEDIUM	FINE	
	3/4"	NO. 4	NO. 10	NO. 40	NO. 200
3/4 - 0.4	42.6	16.8	18.9	10.9	10.8
3/4 - 0.5	50.1	17.5	17.6	8.6	6.2
3/4 - 0.6	56.6	17.6	15.7	6.5	3.6
3/4 - 0.8	67.1	16.5	11.7	3.5	1.2
3/4 - 1.0	75.1	14.4	8.3	1.8	0.4
3/4 - 1.2	81.1	12.2	5.7	0.9	0.1
3/4 - 1.5	87.6	9.0	3.1	0.3	0.0
3/4 - 2.0	93.8	5.1	1.0	0.1	0.0

TABLE 2 BASE COURSE GRADATIONS
MAXIMUM SIZE 3/4 IN.

SAMPLE	PERCENTAGE RETAINED BETWEEN				
	FINE GRAVEL	SAND			SILT OR CLAY
		COARSE	MEDIUM	FINE	
	3/4"	NO. 4	NO. 10	NO. 40	NO. 200
4 - 0.5	0.0	35.2	35.1	17.3	12.4
4 - 0.6	0.0	40.6	36.1	15.1	8.2
4 - 0.8	0.0	50.0	35.7	10.7	3.6
4 - 1.0	0.0	58.0	33.2	7.2	1.6
4 - 1.2	0.0	64.7	29.9	4.7	0.7
4 - 1.5	0.0	72.8	24.6	2.4	0.2
4 - 2.0	0.0	82.4	16.8	0.8	0.0

TABLE 3 BASE COURSE GRADATIONS
MAXIMUM SIZE NO. 4

3/4 - 0.5 would in all probability fail to meet the Corps of Engineers' frost requirement (10) which limits the amount of soil smaller than 0.02 millimeters to 3 per cent by weight.

Base course samples 3/4 - 0.6, 3/4 - 0.8, 4 - 0.8 and 4 - 1.0 satisfy the current specifications (20) of the Indiana Department of Highways for type I (open-graded) base course (Spec. C1002.1, 1952).

Base course samples 3/4 - 0.4, 3/4 - 0.5, 4 - 0.5 and 4 - 0.6 satisfy the current specification (20) of this same agency for type II (dense-graded) base course (Spec. 1002.2, 1952).

Preliminary Compaction Studies

Moisture-density relationships were determined for each of the fifteen base course gradations in accordance with the Corps of Engineers' Specification CE 807.1 (13) which requires compaction in a 6-inch-diameter mold by a 10-pound hammer falling 18 inches for a total of 55 blows on each of five layers. Samples were molded to a compacted height of 4.5 inches. CE Specification 807.1 allowed use of the full gradation in all cases since no material exceeded 3/4-inch in diameter. All compacted samples were discarded after use so as to minimize the effects of aggregate breakage.

Considerable difficulty was encountered in developing these compaction curves particularly for the sands where the effects of bulking make moisture content very critical in the range of moisture near the optimum content. Obvious difficulties arose in attempting to screed a level surface on the compaction cylinder where particles as large as 3/4-inch were involved.

The moisture density relationships for the base course samples

studied are shown in Figures 9 and 10.

In addition, it was decided to attempt determination of the density limits of each base course gradation, that is, the maximum and minimum dry unit weights at which each gradation might exist.

The minimum dry density for each base course gradation was determined by spooning the oven-dried material into a tared 4.5 by 6-inch-diameter cylinder from a height which at no time exceeded two inches. The surface was carefully leveled with a straight edge, taking care not to densify the sample. The tared cylinder and contents were then weighed to the nearest 0.01 pound and the dry unit weight computed from the dimensions of the cylinder. Since the screeding of the cylinder surface left some room for discrepancy, particularly with the gravel samples, three trials were made with each gradation and the average value taken as the minimum attainable density.

An attempt was made to attain a maximum density by vibrating a known weight of material in a lucite cylinder which was secured to the upper tray of a Gilson Mechanical Testing Screen. The apparatus used to determine these vibrated densities is shown in Figures 11 and 12.

The procedure adopted for establishing maximum vibrated densities was as follows:

1. Quantities of oven-dried aggregate were combined to yield 9.00 pounds of the desired gradation, to which enough moisture was added to bring the sample to approximately its optimun moisture content.

2. The moist sample was placed in a lucite cylinder (K of Figure 12), being careful to avoid particle segregation, and tamped lightly to a firm, level surface. The cylinder had previously been mounted in the

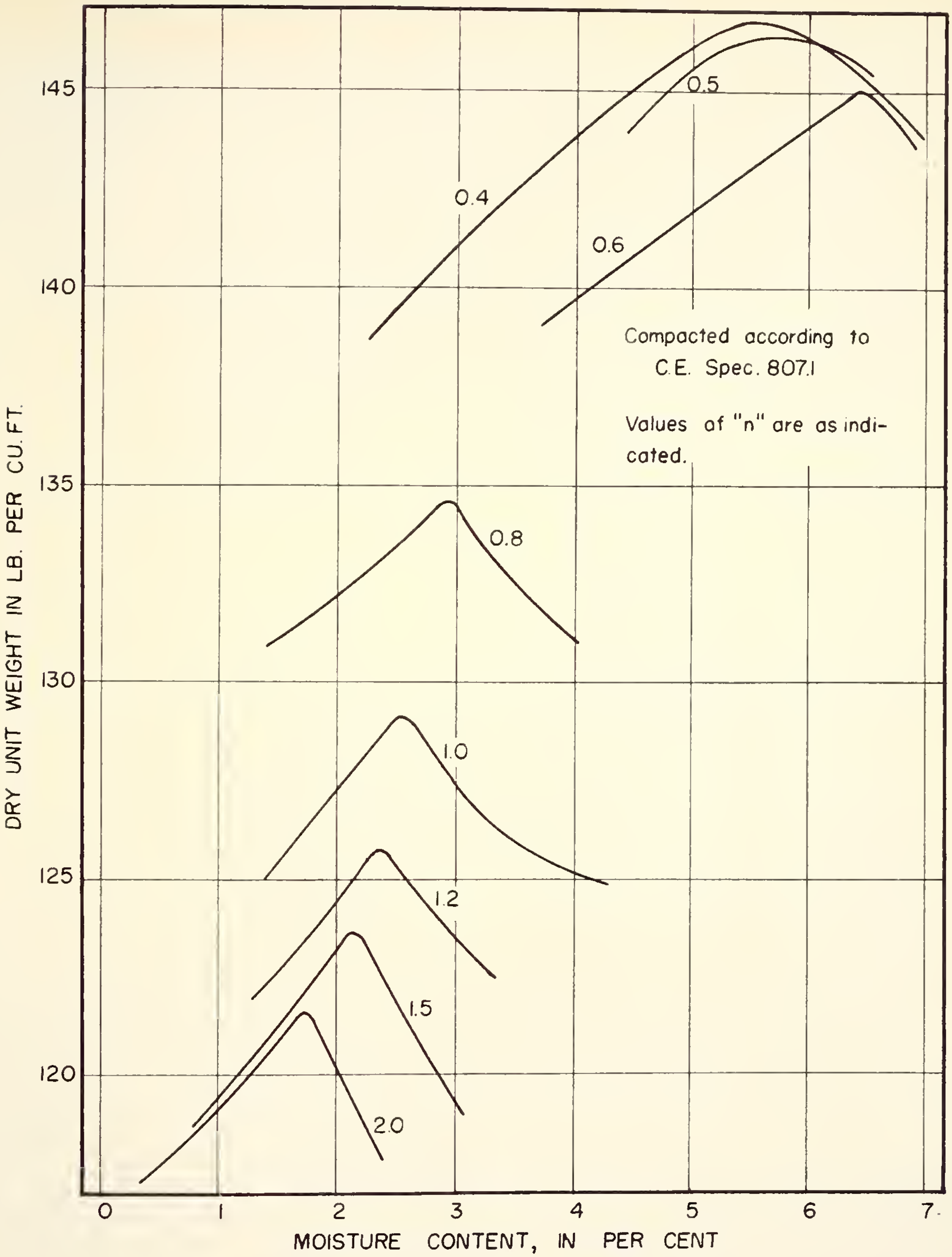


FIG. 9 MOISTURE – DENSITY RELATIONSHIPS FOR BASE COURSE SAMPLES, MAXIMUM SIZE 3/4 IN.

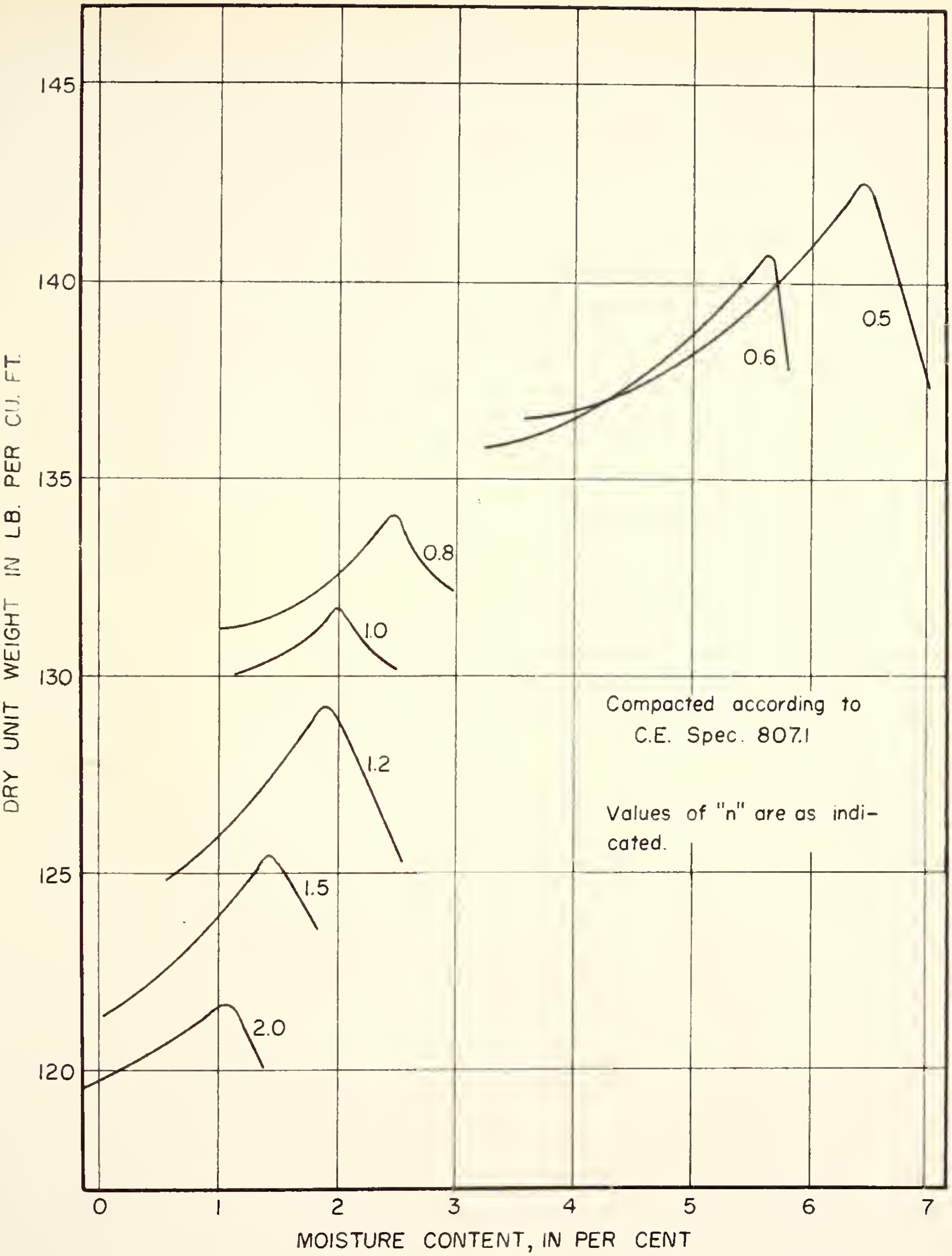


FIG. 10 MOISTURE – DENSITY RELATIONSHIPS FOR BASE COURSE SAMPLES, MAXIMUM SIZE NO. 4

metal base plate (E), supplied with porous stone (H), and placed on a reinforced tray (F) of the Gilson Testing Screen.

3. A 20-pound steel disk (I) was lowered to the sample surface and six 10-pound annular weights (B) placed, one by one, over the vertical extension rod for a total surcharge of approximately eighty pounds.

4. The wooden cushion-frame (J) was then placed between the annular weights and the cylinder wall. Elongated slots in the horizontal members of this frame permitted the insertion of a steel scale to the top surface of the 20-pound steel disk for purposes of measurement.

5. The top plate (A) was then positioned and the four tension rods (C) were bolted into place through the bottom of the reinforced tray.

6. The sample was then vibrated in time increments of ten seconds, vertical measurements being taken after each 10-second inverval by removing the top plate and inserting the steel scale. Each recorded measurement consisted of the average of four, one taken in each quadrant of the cylinder to the nearest 0.02 inch.

7. When the granular sample attained a constant density (as indicated by the same scale measurement for two consecutive intervals) the material was saturated slowly from the bottom through a channel (G) in the base plate by water under a positive hydraulic head.

8. Vibration was then continued in 10-second time intervals until a maximum density was reached.

Each base course gradation was subjected to two vibratory tests of this nature and the greater density value of the two taken as the maximum vibrated density. In nearly every instance, a maximum density was

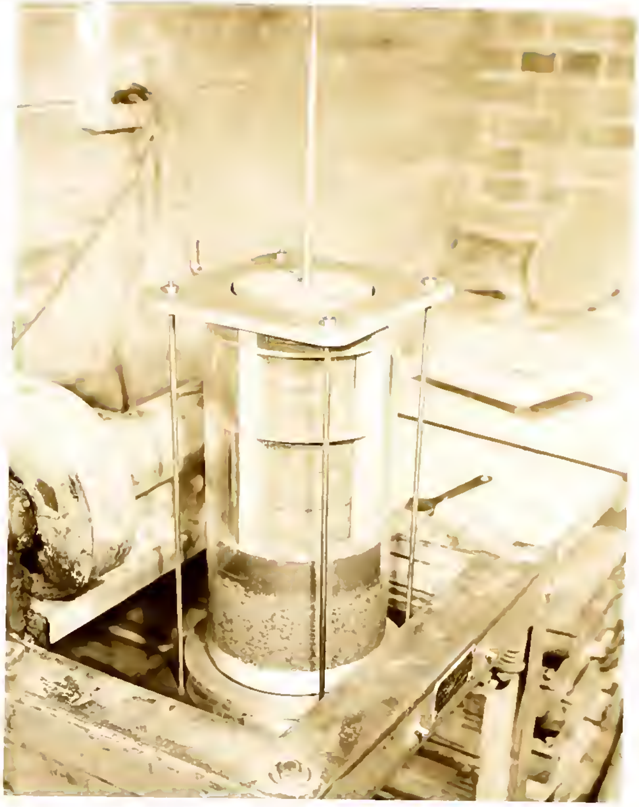
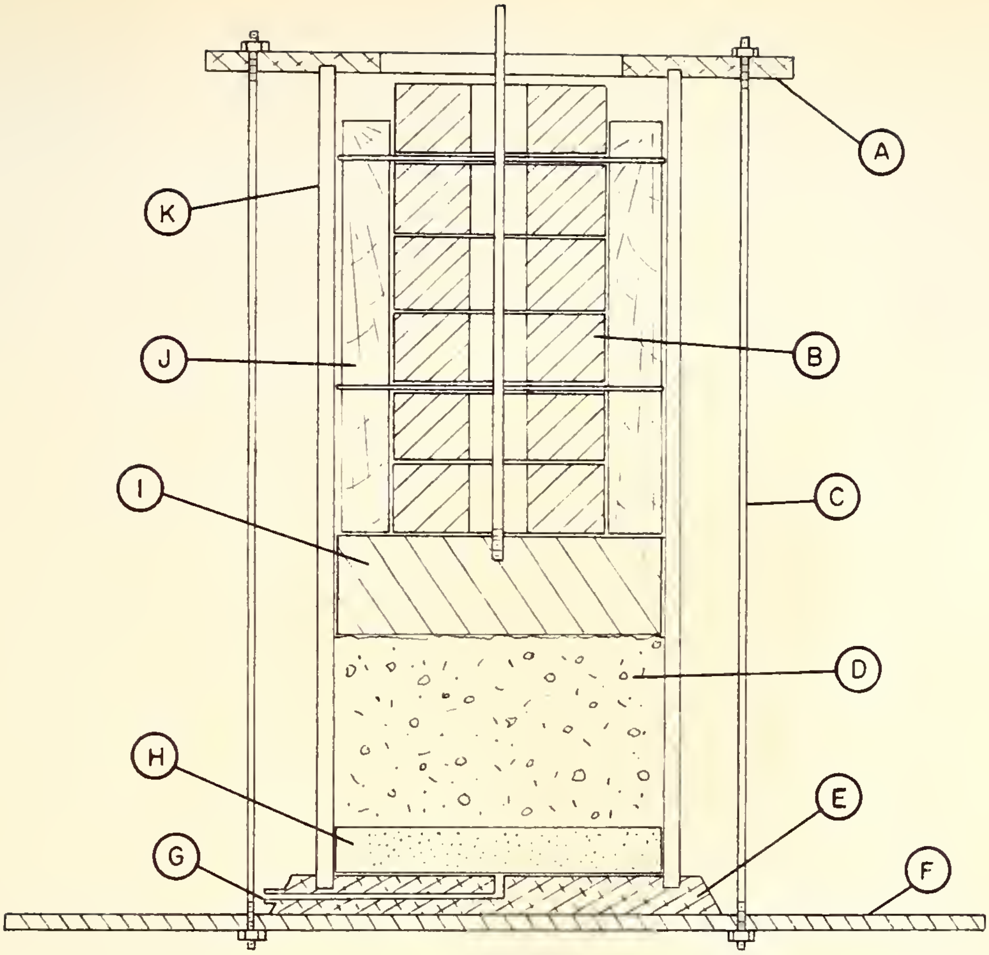


FIG. 11 VIBRATED DENSITY APPARATUS MOUNTED IN THE GIBSON MECHANICAL TESTING SCREEN



A	TOP PLATE	F	REINFORCED TRAY
B	10-LB. ANNULAR WEIGHTS	G	WATER CHANNEL
C	TENSION ROD	H	POROUS STONE
D	BASE COURSE SAMPLE	I	20-LB. STEEL DISK
E	METAL BASE PLATE	J	WOODEN CUSHION-FRAME
K	LUCITE CYLINDER		

FIG. 12 SCHEMATIC DIAGRAM OF VIBRATED DENSITY APPARATUS

attained within 90 seconds and in most cases within 60 seconds. Grain size analyses of vibrated samples indicated that this short term of vibration caused a minimum of aggregate breakage even in the most open-graded samples.

The values obtained for minimum density and maximum vibrated density for the fifteen base course gradations are listed in Table 4.

	¾-0.4	¾-0.5	¾-0.6	¾-0.8	¾-1.0	¾-1.2	¾-1.5	¾-2.0
MINIMUM DENSITY	120.2	121.6	118.9	113.8	109.3	105.9	102.9	100.4
MAXIMUM VIBRATED DENSITY	144.4	140.8	126.1	120.3	117.6	115.8	112.0	110.8

	4-0.5	4-0.6	4-0.8	4-1.0	4-1.2	4-1.5	4-2.0
MINIMUM DENSITY	113.8	121.0	116.8	110.4	107.0	104.8	101.9
MAXIMUM VIBRATED DENSITY	139.1	134.7	129.2	121.7	117.6	114.1	111.4

TABLE 4 MINIMUM DENSITY AND MAXIMUM VIBRATED DENSITY VALUES FOR BASE COURSE GRADATIONS IN LB. PER CU. FT

PROCEDURE

Preparation of Test Samples

Compacted Subgrade

Previous investigations by Havers (17) showed that increasing the compaction of the subgrade had a decided effect on the performance of subgrade-base course combinations subjected to repeated loading. For this study, it was felt that subgrade samples should be compacted to a density level comparable to that which is obtained in practice. Referring to the Engineering Manual of Military Construction (9),

> - - - - Subgrade soils which retain strength when remolded should be prepared in accordance with the following criteria:
> (a) Compacting Fill Sections. All fills, with the exception of those composed of cohesionless sand, should be compacted to not less than 90 percent modified AASHO density. - - - -
> (b) Compacting Cut Sections. The top six inches of subgrade, excluding cohesionless sand, should be compacted to not less than [illegible] percent modified AASHO density. - - - -

It was thus decided to compact subgrade soil samples to 90 per cent of the density obtained following the modified AASHO method (2). Advantage was taken of previous investigations (26 , 17) which indicated that for samples with a large diameter to height ratio a procedure of static compaction from top and bottom would be more likely to yield consistent moisture contents and unit weights between samples and within any single sample than would a method of dynamic compaction.

In Figure 13 appears a pair of curves which relates static compactive pressure to optimum moisture content and maximum dry unit weight of the

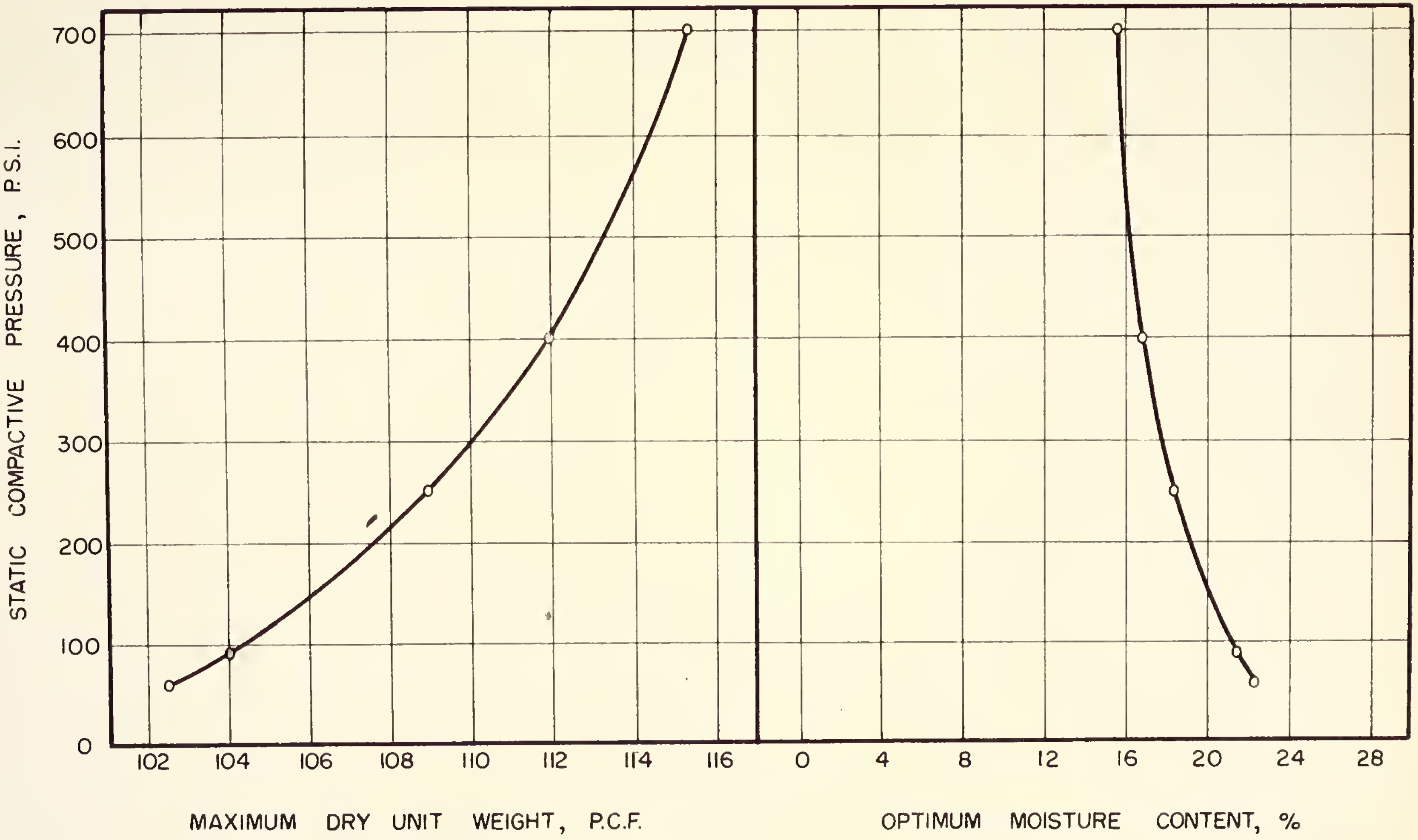


FIG. 13 RESULTS OF STATIC COMPACTION OF THE CROSBY SOIL

After Havers (17)

Crosby soil. These curves are a plot of peak points of a set of moisture-density curves, each moisture-density curve having been developed at a different compactive pressure. All samples used in the preparation of these curves were compacted statically from both ends in a 7-inch inside diameter lucite cylinder to a compacted height of four inches. Figure 13 provided the basis upon which the molded moisture content and compactive pressure were selected for preparation of subgrade samples used in this study.

In order to prepare subgrade samples compacted to the desired height and dry unit weight, it was first necessary to determine the moisture content of the Crosby soil as it existed in storage. On the basis of this initial moisture content, a quantity of soil, about 0.20 pound in excess of that required to form a compacted specimen 7.22 inches in diameter and 4.0 inches in height at a dry unit weight of 106.9 pcf (90 per cent modified AASHO), was weighed into a tared galvanized pan. Distilled water was then incorporated into the soil by means of an electric paint sprayer and continuous troweling (Figure 14) until a sufficient quantity had been added to bring the soil to a moisture content of about 19.5 per cent. Extreme care was exercised to insure that water and soil were well mixed and that no moist clay lumps in excess of about 1/8-inch were allowed to remain unbroken. This mixing process took about 1-1/2 hours. The galvanized pan containing the prepared soil was then covered with a waterproof plastic cloth and placed in a constant humidity room to cure for a period of 24 hours. After curing, the soil was again thoroughly mixed in the galvanized pan and four samples, of approximately 20 grams each, taken for moisture determination.

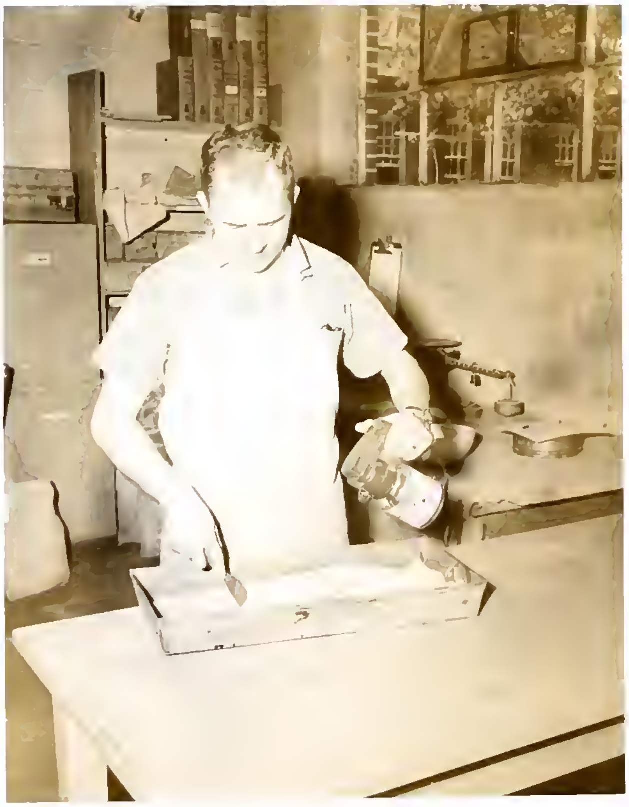


FIG. 14 PREPARATION OF SUBGRADE SOIL

A tared, 7-inch inside diameter lucite cylinder was then placed over a closely-fitting 1-3/4 inch high steel spacer-disk. The prepared Crosby soil, having been removed from the constant humidity room, was then placed into the lucite cylinder in two equal layers each of which was first leveled and then tamped 25 times with a 2-inch-diameter steel tamping rod weighing 5.5 pounds and dropped from a height of approximately four inches. The lucite cylinder, soil, and spacer-disk were then placed on the lower platen of a Riehle lever-type testing machine of 50,000-pound capacity. A steel compaction head, two inches in thickness and machined for a slip fit against the inside of the lucite cylinder, was then inserted and fitted to the upper platen of the testing machine. The soil was then compacted (Figure 15) from both ends at a speed of 0.44 inches per minute for the first three-quarters of the total desired load and at 0.11 inches per minute for the remaining portion of the load. When the desired total load was attained, the direction of motion of the loading head was reversed and the sample thickness measured to .001 inch at a total load of 1000 pounds by means of a dial indicator which was calibrated before each test by placing a machined aluminum cylinder between the loading head and the lower platen of the testing machine.

As the testing of subgrade-base course combinations progressed, however, the inside surface of the lucite cylinders became somewhat scarred. This condition, combined with the subsequent intrusion of soil particles between the compaction head and the cylinder wall altered the frictional characteristics of these surfaces and rendered the above procedure unreliable. Since it was possible to calculate the reading of the calibrated dial indicator which corresponded to the desired sample height, the

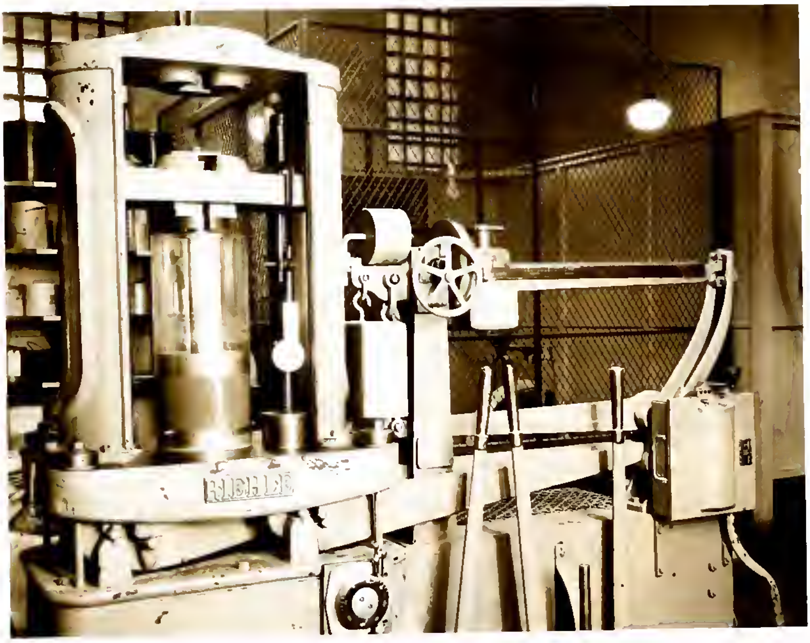


FIG. 15 COMPACTION OF SUBGRADE SAMPLES

basis of subgrade sample preparation was changed to compaction to a given height rather than to a given compactive pressure. This procedure involved compacting the soil to approximately .04 inch in excess of that desired, reversing the direction of motion of the loading head and measuring the sample to .001 inch thickness at 1000 pounds by means of the calibrated dial indicator. Following this procedure, it was possible to compact soil samples to 4.00 ± .02 inches.

After compaction, the steel spacer-disk was removed and the compacted soil and tared cylinder weighed to the nearest .01 pound. A 1-inch thick porous stone was then placed against the bottom of the compacted soil and this assembly placed in a pan of water to the depth of the porous stone. A swell plate was placed on top of the compacted soil and an initial reading taken to the nearest .001 inch. The lucite cylinder was then covered with a moist cloth and a piece of waterproof plastic cloth and allowed to absorb moisture from the bottom for a period of about 50 hours at which time a final swell reading was taken, the swell plate removed from the subgrade surface and the final height of the compacted subgrade computed. The lucite cylinder with soil and porous stone was then fitted into an aluminum base plate and made ready to receive the granular base course material.

Base Course Material

In order to provide a significant basis for comparison of base course samples, it was decided to utilize the concept of relative density rather than compacting samples to some arbitrary percentage of a dynamic compactive test.

For simplicity, Havers (17) compacted samples in terms of a density

ratio, D_r, given by

$$D_r = \frac{d_n - d_o}{d_{100} - d_o}$$

where d_n = dry density, pcf, of compacted specimen,

d_o = dry density, pcf, loosest state from laboratory test for minimum density, and

d_{100} = dry density, pcf, maximum feasible density obtained in the laboratory.

This expression is analogous to that proposed by Lane (25) and is similar to Terzaghi's relative density (31, p.27) but has the advantage that specific gravity of solids need not be determined. The two yield results which are quite similar, as shown by Lane (25). The concept of density ratio, as described above, was used in this study.

Involved in such a concept as density ratio are the procedures used to define the density limits, d_o and d_{100}. The minimum density value obtained for each base course gradation, described under Preliminary Compaction Studies of Base Course Materials, was taken as d_o, dry density in the loosest state. Based on a limited number of auxillary tests, d_{100} was, in each case, taken as the average of two values, the maximum vibrated density already described (page 56) and the dry unit weight of the material at its optimum moisture content when compacted according to CE Spec. 807.1 (pp. 50, 51). A discussion of the selection of these values for density limits is presented later in this text.

It was decided to compact base course samples to a density ratio of 90 per cent, which might be considered a minimum requirement in order to limit objectionable consolidation and settlement under heavy vibratory

loading (7). For each base course sample, the weight of material, W, required to form a specimen 4.0 inches in height at a density ratio of 0.90 was computed from the diameter, ϕ, of the lucite cylinder and the definition of density ratio by the following formula:

$$W = (\pi/1728)\phi^2\left[d_o + 0.9(d_{100} - d_o)\right]$$

The base course sample was then prepared in two equivalent parts by combining the various aggregate size fractions in the proper proportions. Enough water was then added to each part to bring the total sample to a moisture content slightly in excess of the optimum content indicated by dynamic compaction according to CE Spec. 307.1. This procedure seemed to cause a minimum of aggregate segregation during placing. At the completion of the subgrade soaking period, the base course material was carefully placed, in two equal layers, onto the compacted subgrade surface inside the lucite cylinder. Placement was accomplished with a long-handled spoon taking care to place the material as uniformly as possible. Each layer was then compacted to a height of two inches by first, tamping lightly with a 2-inch diameter, 5.5 pound tamping rod and then by applying several sharp blows with a leather mallet to a compaction head placed on the base course surface. This compaction head consisted of an aluminum disk one inch in thickness and 6.97 inches in diameter attached to 1-1/4-inch diameter aluminum shaft to which the blows of the leather mallet were applied. After the compaction head had been removed, a swell plate was inserted and the final compacted height of the sample was checked with a dial indicator mounted on a tripod.

SAMPLE	DRY UNIT WEIGHT (PCF) OBTAINED BY			d_0 (PCF)	d_{100} (PCF)	$D_r = 0.90$* (PCF)
	POURING LOOSE INTO MOLD	VIBRATION	DYNAMIC COMPACTION (MAXIMUM)			
3/4 - 0.4	120.2	144.4	146.8	120.2	145.6	143.1
3/4 - 0.5	121.6	140.8	146.4	121.6	143.6	141.4
3/4 - 0.6	118.9	126.1	145.0	118.9	135.6	133.9
3/4 - 0.8	113.8	120.5	134.6	113.8	127.4	126.0
3/4 - 1.0	109.3	117.6	129.1	109.3	123.4	122.0
3/4 - 1.2	105.9	115.8	125.7	105.9	120.8	119.3
3/4 - 1.5	102.9	112.0	123.6	102.9	117.8	116.3
3/4 - 2.0	100.4	110.8	121.6	100.4	116.2	114.6

* Computed by formula, $D_{90} = d_0 + 0.9(d_{100} - d_0)$

TABLE 5 RESULTS OF DENSITY TESTS PERFORMED ON GRAVEL BASE COURSE SAMPLES

SAMPLE	DRY UNIT WEIGHT (PCF) OBTAINED BY			d_0 (PCF)	d_{100} (PCF)	$D_r = 0.90$* (PCF)
	POURING LOOSE INTO MOLD	VIBRATION	DYNAMIC COMPACTION (MAXIMUM)			
4 - 0.5	113.8	139.1	142.6	113.8	140.8	138.1
4 - 0.6	121.0	134.7	140.7	121.0	137.7	136.0
4 - 0.8	116.8	129.2	154.1	116.8	131.6	130.1
4 - 1.0	110.4	121.7	131.8	110.4	126.8	125.2
4 - 1.2	107.0	117.6	129.3	107.0	123.4	121.8
4 - 1.5	104.8	114.1	125.5	104.8	119.8	118.3
4 - 2.0	101.9	111.4	121.7	101.9	116.6	115.1

* Computed by formula, $D_{90} = d_0 + 0.9(d_{100} - d_0)$

TABLE 6 RESULTS OF DENSITY TESTS PERFORMED ON SAND BASE COURSE SAMPLES

Combined Sample

The lucite cylinder containing the compacted subgrade and base course was then placed in the repetitive loading machine. The loading head was inserted and a cover plate fitted to the top of the cylinder. The cylinder and base plate were then shifted until centered under the piston rod of the air cylinder which was then coupled to the extension rod of the loading head and minor adjustments made to bring the loading head in light contact with the base course surface and center it inside the lucite cylinder. The base plate was then secured to the testing machine and the tie rods holding down the cover plate tightened. At this point, a seating load of about 50 psi was applied through the loading head to the base course surface. The water line leading to the standpipe and reservoir was then connected to the base plate.

Water was added to the base course through a small opening in the cover plate until the free water level existed at slightly above the base course surface. Previous investigations (17) had indicated that pumping of the base course would not occur unless the water level in the lucite cylinder was at or slightly above the base course surface. The valves leading to the standpipe and reservoir were then opened allowing water to flow through the porous stone and discharge to the atmosphere. This operation was done with some care so as not to cause erosion of the subgrade immediately above the porous stone. When air bubbles were no longer visible in the discharge, the discharge valve was closed and a static reservoir pressure of approximately 2-3/4 psi was allowed to remain on the bottom of the subgrade for a period of about 12 hours. At the end of this time, water was allowed to briefly flow through the

porous stone to discharge any additional accumulation of air. The discharge valve and the valve leading to the standpipe were then closed, in that order, so that any water was prevented from leaving or entering the subgrade-base course system through the bottom. The sample was now ready for testing.

Tests Performed on Individual Specimens

The repetitive loading apparatus had been preset to deliver a 25 psi load of 0.3 second duration to the base course surface every four seconds. A total of 40,000 repetitions of this load was applied to each subgrade-base course combination. Periodic measurements of the permanent and elastic deformations of the system were made throughout each test by means of a linear variable differential transformer already described under the section on TESTING EQUIPMENT. The step by step procedure used for deflection measurement was as follows:

1. The bridge circuit of the BL-320 Universal Analyzer was balanced by removing the transformer output leads from the bridge and inserting, in their place, leads to an AC voltmeter. The balance knobs of the analyzer were then adjusted until there was zero voltage across the bridge. This operation was necessary so that movements of the oscillograph pen would produce a linear calibration with vertical movements of the core of the transformer. This had to be done only once for the entire series of fifteen tests.

2. The transformer core was then moved up into the coil, the output from the transformer placed back across the bridge of the analyzer, and the core moved into its center or null position by manipulation of the micrometer barrel. This point was reached when the voltmeter, still

across the bridge, again showed zero voltage. At this point, the micrometer was read and the voltmeter was disconnected.

3. The knob controlling the AC gain on the analyzer was adjusted so that the oscillograph pen when starting from one margin of the oscillograph, would travel to the opposite margin when the core of the transformer was moved downward a distance of 0.04 inch. For convenience a calibration of one division on the oscillograph equal to .001 inch vertical movement of the transformer core was sought. During this calibration, the core could at any time be returned to its null position by moving the micrometer to the null reading previously taken, and was done so just prior to the start of a test.

4. The test was then started by closing the main-line switch and turning on the analyzer. Elastic and permanent deflections were recorded as a time-deflection curve on the oscillograph (Figure 4). At any time during the test, deflections of the subgrade-base course system could be measured by a consideration of direct readings from the oscillograph and the initial calibration of the recording equipment. As the total deflection approached 0.04 inch, the transformer core was moved back toward its null position by turning the micrometer screw and the new micrometer reading recroded. The difference in the two readings was then added to the permanent deflection recorded by the oscillograph pen. The analyzer and recorder were run continuously through approximately 100 load repetitions and then monitored as deflection measurements were desired.

As each test was completed, any material that had been pumped above the surface of the base course was removed, dried, weighed and subjected to a sieve analysis. For the more open-graded samples, note was taken

of the extent of subgrade intrusion into the base course, if discernable. The entire sample was then extruded by means of a screw-jack and grain size analyses were performed on the base course to determine any changes which may have occurred in its gradation as a result of the repetitive loading test.

RESULTS

The results of the repetitive loading tests on the fifteen subgrade-base course combinations considered in this study are summarized in the following figures and tables. In Figures 16 and 17, the total deflection (permanent plus elastic) of each sample is plotted as an ordinate against the logarithm of the number of load applications. In each case, deflection readings were not commenced until after 100 applications of the 25 psi load in order to allow for adequate seating of the loading head. For this reason, a zero deflection is shown at 100 load repetitions in Figures 16 and 17.

At the termination of each test, grain size analyses were performed on different portions of the base course sample or, as in a few cases, on the base course as a whole, to determine any change in gradation which may have occurred as a result of the repetitive loading. In addition, a grain size analysis was performed on any material which had been pumped to the top surface of the base course. From these data, the increase in minus No. 200 material in the base course, the weight of minus No. 200 material moved to the top of the base course and the increase in minus No. 200 material above the subgrade were computed and recorded in Tables 7 and 8 . The grain size distribution curves of base course samples are presented in the appendix.

The transparent lucite cylinder which was used to confine the subgrade-base course samples permitted a visual inspection of specimens at

any time during the testing procedure. The intrusion of subgrade soil into the base course was observed in five test samples. In the three most open-graded sands (4 - 1.2, 4 - 1.5, and 4 - 2.0), subgrade intrusion appeared to be only slight, not exceeding 1/4 - 1/2 inch in extent, while intrusion to approximately 1 1/2-inches was observed in two of the open-graded gravels (3/4 - 1.5 and 3/4 - 2.0). The observations were verified by grain size analyses performed on the base courses after testing. The pumping of fine soil to the base course surface was observed in the case of three specimens with dense-graded gravels (3/4 - 0.4, 3/4 - 0.5 and 3/4 - 0.6) but in only one case where a sand base course was used (4 - 0.5), and then only to a small degree.

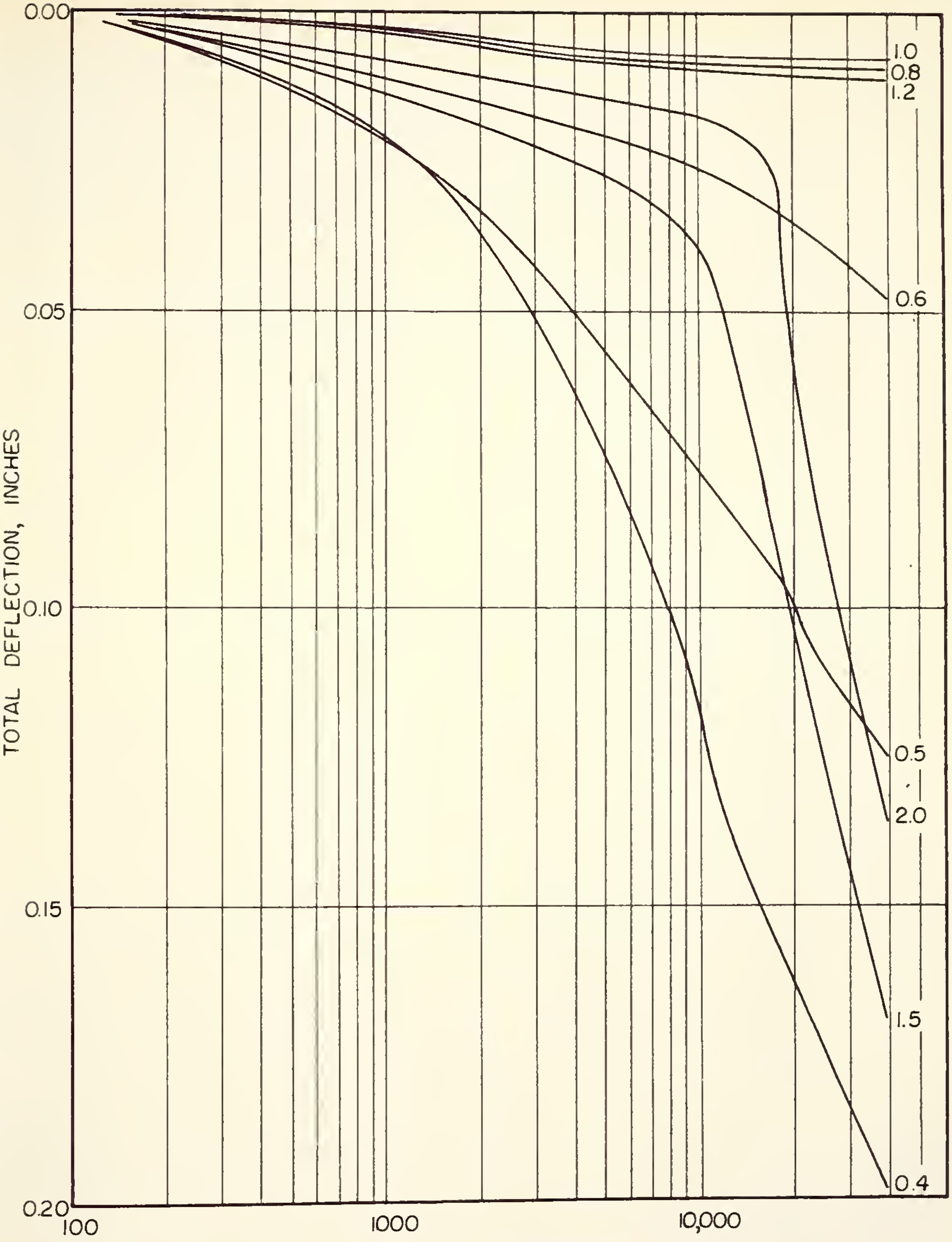


FIG. 16 DEFLECTION CURVES FOR SAMPLES WITH GRAVEL BASE COURSE

SAMPLE	PUMPED MATERIAL			WEIGHT OF MINUS NO. 200 IN BASE			INCREASE IN MINUS NO. 200 MTRL. ABOVE SUBGRADE, GR.
	TOTAL WEIGHT PUMPED GR.	WEIGHT MINUS NO. 200 MTRL. PUMPED, GR.	PER CENT MINUS NO. 200	BEFORE GR.	AFTER GR.	NET INCREASE GR.	
3/4 - 0.4	206	143	69.4	676	539	−137	6
3/4 - 0.5	121	103	85.1	378	290	−88	15
3/4 - 0.6	81	70	86.4	204	148	−56	14
3/4 - 0.8	0	0	—	64	68	4	4
3/4 - 1.0	0	0	—	22	25	3	3
3/4 - 1.2	0	0	—	4	39	35	35
3/4 - 1.5	0	0	—	0	262	262	262
3/4 - 2.0	0	0	—	0	151	151	151

TABLE 7 TEST RESULTS FOR SAMPLES WITH GRAVEL BASE COURSE

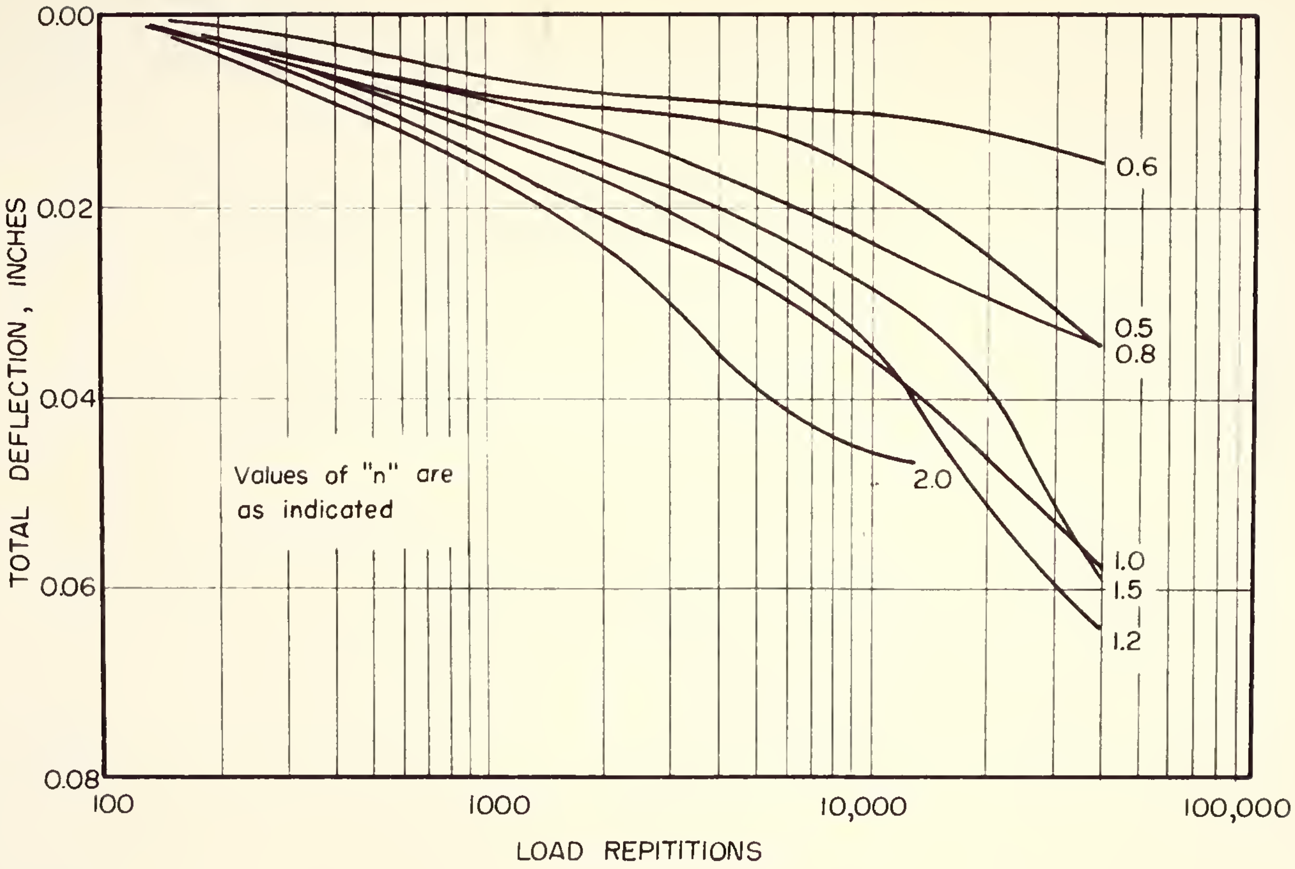


FIG. 17 DEFLECTION CURVES FOR SAMPLES WITH SAND BASE COURSE

SAMPLE	PUMPED MATERIAL			WEIGHT OF MINUS NO.200 IN BASE			INCREASE IN MINUS NO.200 MTRL. ABOVE SUBGRADE, GR.
	TOTAL WEIGHT PUMPED GR.	WEIGHT MINUS NO.200 MTRL. PUMPED, GR.	PER CENT MINUS NO.200	BEFORE GR.	AFTER GR	NET INCREASE GR.	
4 - 0.5	15	8	53.4	736	723	−13	−5
4 - 0.6	0	0	—	481	478	3	3
4 - 0.8	0	0	—	200	204	4	4
4 - 1.0	0	0	—	82	80	−2	−2
4 - 1.2	0	0	—	36	51	15	15
4 - 1.5	0	0	—	9	33	17	17
4 - 2.0	0	0	—	0	24	24	24

TABLE 8 TEST RESULTS FOR SAMPLES WITH SAND BASE COURSE

DISCUSSION

Introduction

In order to properly evaluate the test results presented in the preceding pages, it is essential that the reader be familiar with the framework of test conditions in which the repeated load tests were performed, the degree of precision obtained in the preparation of samples and the accuracy with which results were measured.

Conditions of Testing

Applied Load

Each subgrade-base course sample was subjected to 40,000 repetitions of a 25 psi load applied at the base course surface for a duration of 0.3 seconds and at an interval of 4.0 seconds between load applications. The 0.3-second load duration was short enough to approximate the sinusoidal-type load-time characteristics of a deflecting pavement slab which accompany the passage of vehicular traffic, and was sufficiently long to allow development of the full 25 psi load before release. A 4.0-second interval was also more than adequate to allow complete release of one load before the application of another (see Figure 4, p. 33). A load intensity of 25 psi may be somewhat in excess of that generally measured under a rigid pavement which is at least eight inches in thickness (5 , p. 45), but it was chosen in order to magnify differences in performance between different subgrade-base course samples. Such a load

intensity would not, however, be unreasonable for the construction period, which time may be critical in many respect from the standpoint of overall pavement performance and quality.

Subgrade Compaction

All subgrade samples were compacted to approximately 90 per cent of the maximum dry unit weight obtained by the modified AASHO test (2). The Corps of Engineers, U. S. Army (9) requires this degree of compaction, as a minimum, for both cut and fill sections with the exception of those composed of cohesionless sand. This requirement, however, may be slightly below the average degree of compaction required by the various state highway departments.

Base Course Compaction

All base course samples were compacted to a density ratio of 90 per cent which involved determination of the values d_o and d_{100} (described in section on PROCEDURES) for each of the fifteen different aggregate gradations. The values obtained for minimum density, d_o, can be considered as highly reliable since the procedure followed is one that is well accepted (17, 25) and in no case did the weights obtained from successive trials with the same sample vary by more than 0.06 pounds. Determination of maximum density values, d_{100}, however, was admittedly quite arbitrary, being, in each case, an average of the maximum modified AASHO dry unit weight (according to CE Spec. 807.1) and a maximum dry unit weight obtained by a procedure of mechanical vibration.

The maximum dry unit weight obtained by dynamic compaction according to CE Spec. 807.1 has been, in the past, taken for d_{100} (17), but in this case it was suspected to give values somewhat higher than the

true maximum density, particularly for the more open-graded materials. The dynamic compaction used is a rather severe test, causing a considerable amount of aggregate breakage such that the densities obtained are not those of the original gradation but, rather, of some new gradation resulting from an alteration of the original material. Shelburne (28) showed that the breakage of aggregate due to mixing, rolling, traffic, and the Los Angeles Abrasion Test results in gradations which tend to approach Fuller's curve of maximum density. This same tendency could be presumed to exist in the dynamic compaction test. If so, it would be reasonable to assume that one would obtain unit weights in the dynamic test which would be higher than had no breakage occurred. Since the dynamic compaction test performed according to CE Spec. 807.1 applies considerable agitation and compactive effort to a granular sample it is conceivable that unit weights approximating a maximum density could be obtained if no breakage were to occur, and that densities are obtained in this test which are probably higher than d_{100} for the unaltered material. This supposition is supported, in part, by figure 18 which indicates that a slight change in the gradation of an open-graded material with the addition of a small amount of fines (which occur in the dynamic test) may change the density that can be obtained with the material by a considerable amount.

As a matter of interest, grain size analyses were performed on two samples of each base course gradation after having been subjected to dynamic compaction according to CE Spec. 807.1. The results of these analyses are shown in Table 9 as a tabulation of increase in per cent finer than the various sieve sizes against base course gradation. By inspection of Table 9, it can be seen that the greater aggregate break-

MAXIMUM AGGREGATE SIZE, $\frac{3}{4}$ INCH								
INCREASE IN PER CENT FINER THAN	VALUES OF "n"							
	0.4	0.5	0.6	0.8	1.0	1.2	1.5	2.0
3/4 IN.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1/2 IN.	——	——	——	0.4	5.6	9.7	12.0	9.9
3/8 IN	2.3	0.2	4.2	6.1	12.1	15.5	19.2	11.9
NO. 4	3.6	3.1	6.0	8.3	12.1	15.7	19.6	11.9
NO. 10	1.1	0.5	2.1	4.1	6.3	8.1	9.3	3.7
NO. 20	2.3	2.4	2.9	3.3	5.2	6.1	6.9	2.6
NO. 40	2.3	2.5	2.7	2.9	4.7	5.2	6.1	2.1
NO. 80	1.8	2.2	2.3	2.6	4.5	4.5	5.4	2.0
NO. 200	1.4	1.6	1.6	1.8	3.7	3.7	4.8	1.6
MAXIMUM AGGREGATE SIZE, NO. 4								
NO. 4		0.0	0.0	0.0	0.0	0.0	0.0	0.0
NO. 10		1.0	2.8	1.8	2.6	3.9	5.4	9.6
NO. 20		3.4	3.8	4.0	5.2	5.9	6.1	7.2
NO. 40		4.0	4.9	5.3	5.1	5.8	4.0	4.8
NO. 80		3.9	4.3	4.6	4.8	5.0	2.7	3.6
NO. 200		1.2	1.6	2.0	2.2	2.9	1.6	2.7

TABLE 9 TABULATION OF AGGREGATE BREAKAGE IN THE DYNAMIC COMPACTION TEST

age does occur in the more open-graded materials with the production, in each case, of amounts of minus No. 200 material in excess of 1.2 per cent, and in most cases more.

The compaction curve shown in Figures 9 and 10 for the open-graded samples are of interest from a research standpoint but have little significance from the viewpoint of field compaction control. In the case of most of the more open-graded materials the moisture-density relationship was not well defined and was confined to narrow limits of unit weight and moisture content. Optimum moisture content has little significance for these materials from the standpoint of compaction control since any water in excess of that required to coat the aggregate surface is not retained but merely passes through the sample. These observations plus the aggregate breakage encountered and the inherent difficulties involved in developing curves for this sort of material strongly support the rather well-accepted contention that such a dynamic compaction test is of little value in specifying compaction requirements for fairly clean granular materials.

The vibrated density test which was used, on the other hand, was believed to give values which were lower than true maximum density. It had been pointed out (17) that a greater surcharge and intensity of vibration would most likely result in higher unit weights. One advantage of this test, however, lay in the fact that a maximum density could, in most cases, be obtained within 60 seconds of vibration. Grain size analyses performed on samples which had been vibrated revealed that this short term of vibration resulted in only minor amounts of aggregate breakage. A 9.00 pound sample of base course aggregate was prepared according to the altered gradation, $4 - 1.2_a$, resulting

from dynamic compaction according to CE Spec. 807.1. This sample was vibrated until a maximum dry unit weight of 126.7 pcf was obtained. Comparing this value with 117.6 pcf obtained by vibrating the unaltered sample 4 - 1.2 (Table 6, p. 67), and 129.3 pcf resulting from dynamic compaction (Table 6, p. 67) of the same sample indicates that the actual density values obtained by vibration and dynamic compaction on the same gradation of material may not be as far apart as indicated by Tables 5 and 6, where their true relationship is masked by aggregate breakage in the dynamic test.

The method of determining d_{100}, thus, was arbitrarily chosen and may be subject to some criticism. Fortunately, however, the degree of base course compaction was found to have little direct effect on the values of the measured variables in such a test (17). It was observed in this investigation that the principle effect of a lower degree of base course compaction was to cause a greater initial settlement within the first 100 or 200 load applications, presumedly, within the base course primarily. Thus, any deformation or lack of deformation which might have resulted from discrepancies between base course densities as compacted and true density ratios of 90 per cent could very well have been masked by the fact that deflection readings were not plotted until 100 repetitions of the 25 psi load had been applied.

A comparison of the various density values which were determined experimentally is shown in Figure 18. It is interesting to note how critical, with respect to attainable unit weights, a small change in aggregate gradation may be within certain gradational ranges.

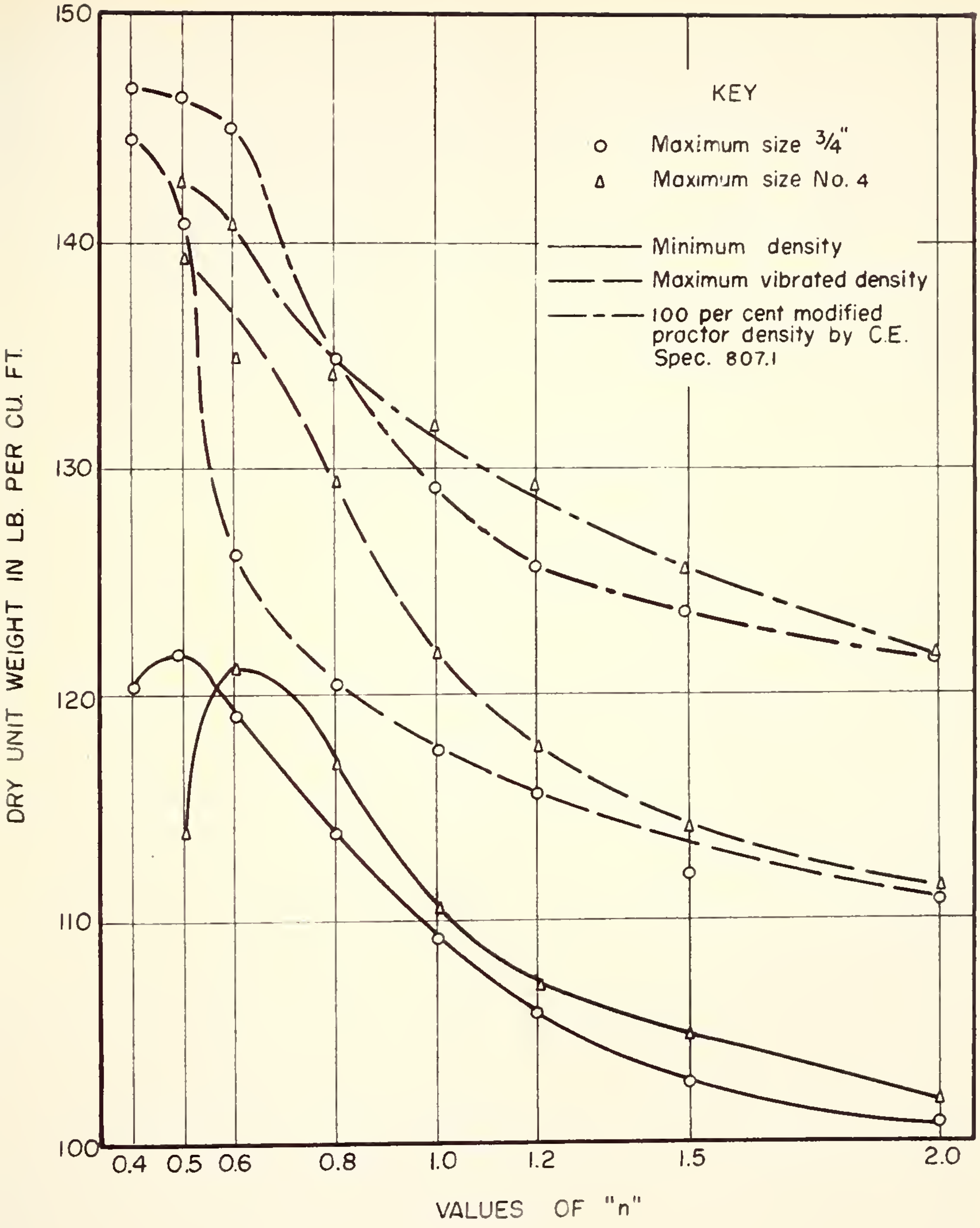


FIG. 18 COMPARISON OF DENSITY VALUES FOR DIFFERENT BASE COURSE GRADATIONS

Sample Preparation

Compacted Subgrade

Before any tests were begun, enough soil was brought into the laboratory and processed to supply the needs of the entire investigation. Extreme care was taken to insure adequate mixing of the soil as checked by numerous Atterberg Limits. The procedure of static compaction which was adopted permitted good reproduction of unit weight between samples (see Table 10).

TABLE 10. VARIATION BETWEEN COMPACTED SUBGRADE SAMPLES

SAMPLE	COMPACTED DRY UNIT WEIGHT, PCF	PER CENT MODIFIED AASHO
3/4 - 0.4	107.72	90.67
3/4 - 0.5	107.92	90.84
3/4 - 0.6	106.43	89.59
3/4 - 0.8	106.91	90.00
3/4 - 1.0	106.41	89.57
3/4 - 1.2	106.53	89.67
3/4 - 1.5	106.73	89.84
3/4 - 2.0	107.07	90.13
4 - 0.5	106.96	90.03
4 - 0.6	106.39	89.55
4 - 0.8	107.04	90.10
4 - 1.0	106.49	89.64
4 - 1.2	107.38	90.39
4 - 1.5	107.28	90.30
4 - 2.0	106.70	89.81

Base Course Sample

All base course aggregate was carefully separated, washed and stored before any testing was begun. Extreme care was taken in the batching of test samples, weighing all component size fractions to the nearest gram. Any aggregate which had previously been used in a way that might cause breakage or crushing was discarded. Each base course sample was prepared and placed onto the compacted subgrade in two separate portions to help attain uniformity throughout the sample. Particle segregation was reduced by mixing each sample to a moisture content near its optimum and by careful placement in the lucite cylinder. Even with these precautions, however, it was not possible to insure a high degree of uniformity within individual samples, particularly in the larger sized gravels, because of the nature of the material. Segregation during placing appeared to be at a minimum in the dense-graded materials which contained appreciable fines but was unavoidable in the more open-graded mixtures. Considerable segregation also occurred during the performance of repeated loading tests as indicated by observation and grain size analyses performed on different components of the same base course after testing (see appendix).

In each case, it was possible to compact the gravel base course samples to approximately the desired height, but in no case could the sand base course sample be compacted to the desired 4-inch thickness indicating that the values of d_{100} approximated for the sands in Table 6 may be too high.

TABLE 11. VARIATION BETWEEN COMPACTED
BASE COURSE SAMPLES

SAMPLE	COMPACTED DRY UNIT WEIGHT, PCF	DENSITY RATIO
3/4 - 0.4	143.5	0.914
3/4 - 0.5	141.5	0.904
3/4 - 0.6	133.8	0.892
3/4 - 0.8	126.1	0.908
3/4 - 1.0	121.8	0.887
3/4 - 1.2	119.5	0.911
3/4 - 1.5	116.4	0.903
3/4 - 2.0	114.3	0.880
4 - 0.5	135.8	0.813
4 - 0.6	134.9	0.832
4 - 0.8	129.4	0.851
4 - 1.0	124.0	0.830
4 - 1.2	120.3	0.811
4 - 1.5	116.8	0.798
4 - 2.0	113.6	0.794

Measurement of Test Results

Of the values measured during and immediately after each repeated load test, the deflection of the subgrade-base course systems were considered to be the most reliable. All that was required was a linear calibration between vertical movements of the transformer core and the pen of the oscillograph recorder. This calibration was checked frequently during each test and the equipment was at no time found to require ad-

justment. The recorded deflections, then, were dependent only upon the performance of the test specimens acting under the imposed conditions. No difficulty was encountered in maintaining the magnitude, duration and frequency of loading constant from test to test or for the duration of any single test. Chance variation in the preparation of subgrade samples and the difficulties already discussed in connection with the placement of base course samples undoubtedly account for a large part of the apparent inconsistencies in the test results. Since no duplicate samples were tested, it is only possible to guess at the reproducibility of results. Havers (17), in tests similar to these, indicated that in some subgrade-base course combinations which showed small deflections, discrepancies of 20 per cent may result.

The measurement of the weight of soil pumped to the surface of the base course was also considered to be highly reliable as most of this material was contained on top of the loading head and around the sides between the loading head and the cylinder wall. Observations made during testing permitted a visual measurement of the movement of the soil.

Considered the least reliable of the measured results was the increase in minus No. 200 material in the base course. The validity of this measurement depended to a large extent on the care that was taken in separating the base course aggregate from the subgrade soil after extrusion of the tested specimen from the lucite cylinder. This act was comparatively easy to perform in the case of specimens with sand base courses where the interface was fairly well defined. In the case of specimens with gravel base courses, however, separation was more difficult and somewhat arbitrary in a few cases, particularly where subgrade soil had been extensively intruded into the more open-graded bases. Aggregate

degradation during testing might contribute to erroneous conclusions about the source of any increase in minus No. 200 soil in the base course. This problem, however, was considered to be of little importance in light of a number of tests performed by Havers (17) from which he estimated that aggregate degradation in the single-acting type of test where deflections were small would be slight and occur to about the same extent in dense-graded samples as in those with a more open gradation.

Interpretation of Results

The transparent lucite cylinder which was used to contain the test specimens allowed visual inspection of performance while the repeated load tests were in progress and provided a means of observing the intrusion of subgrade soil into the base course and the movement of fine soil within the base course to the base course surface. These observations were, in a general way, substantiated by deflection measurements made during each test (Figures 16 and 17) and by values measured immediately after the completion of each test (Tables 7 and 8).

Test specimens with gravel base courses exhibited pumping of fine soil to the surface of the three denser-graded base course samples and intrusion of subgrade soil into the three most open-graded (only slightly in one case). As pointed out by Havers (17), each of these two failure conditions tended to produce a characteristic type of load-deflection curve when total deflections were plotted as ordinates against the logarithm of load repetitions as abscissa. If substantial intrusion of subgrade soil occurred, the deflection curve remained, initially, rather flat and then sloped downward abruptly. This trend is seen in Figure 16 for samples 3/4 - 1.5 and 3/4 - 2.0. If, on the other hand, no intrusion

occurred but substantial material was pumped to the base course surface, the curve showed deflections which were, initially, greater than those recorded for test specimens with the more open-graded bases in which pumping either occurred to a lesser extent or not at all. This trend may be seen in samples 3/4 - 0.4 and 3/4 - 0.5 of Figure 16. Some specimens, however, with base course samples of an intermediate gradation exhibited neither pumping nor subgrade intrusion and resulted in smaller subgrade-base course deflections. A plot of the deflections occurring in each specimen at different repetitions of load (Figure 19) indicates that there may be an optimum range of gradation which, when compacted to a high relative density over a good subgrade, will result in minimum deflections of the subgrade-base course system.

In all cases when specimens with gravel base courses were tested, a good correlation seemed to exist between large deflections and the maladies of base course pumping and subgrade intrusion, and it might be concluded that if a base course gradation could be selected that would resist these latter two effects, deflection of the subgrade-base course system could be held to a minimum in any given situation and pavement cracking and faulting resulting from lack of foundation support might be retarded.

Table 7 indicates minor increases in minus No. 200 soil above the subgrade for base course samples that experienced no visible intrusion of subgrade soil (samples 3/4 - 0.4 through 3/4 - 1.0). This minus No. 200 material was believed to reflect the difficulties encountered in separating the base course sample from the subgrade soil. One rather obvious inconsistency is that sample 3/4 - 2.0 showed smaller deflections and

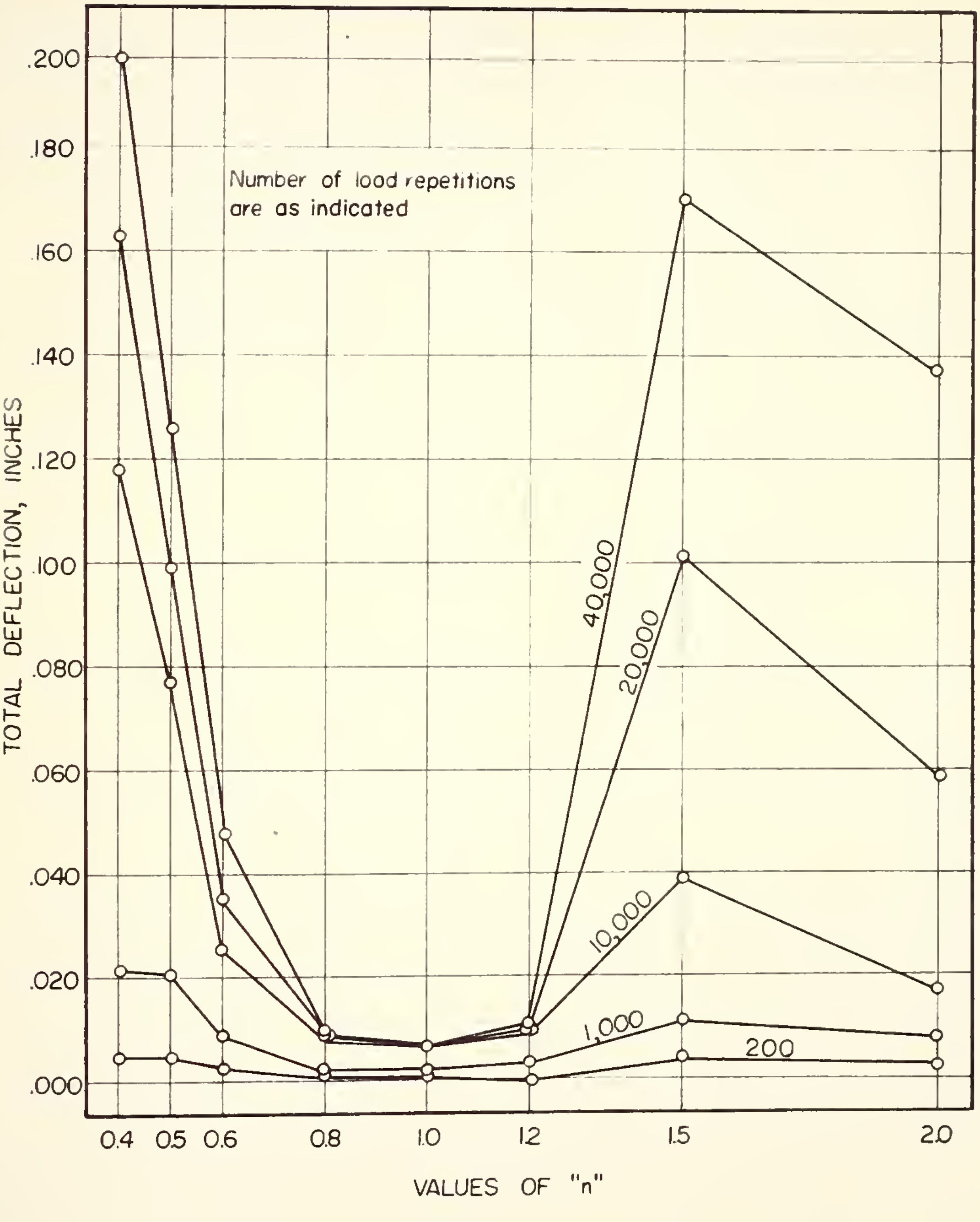


FIG. 19 DEFLECTION VERSUS GRADATION FOR SAMPLES WITH GRAVEL BASE COURSE

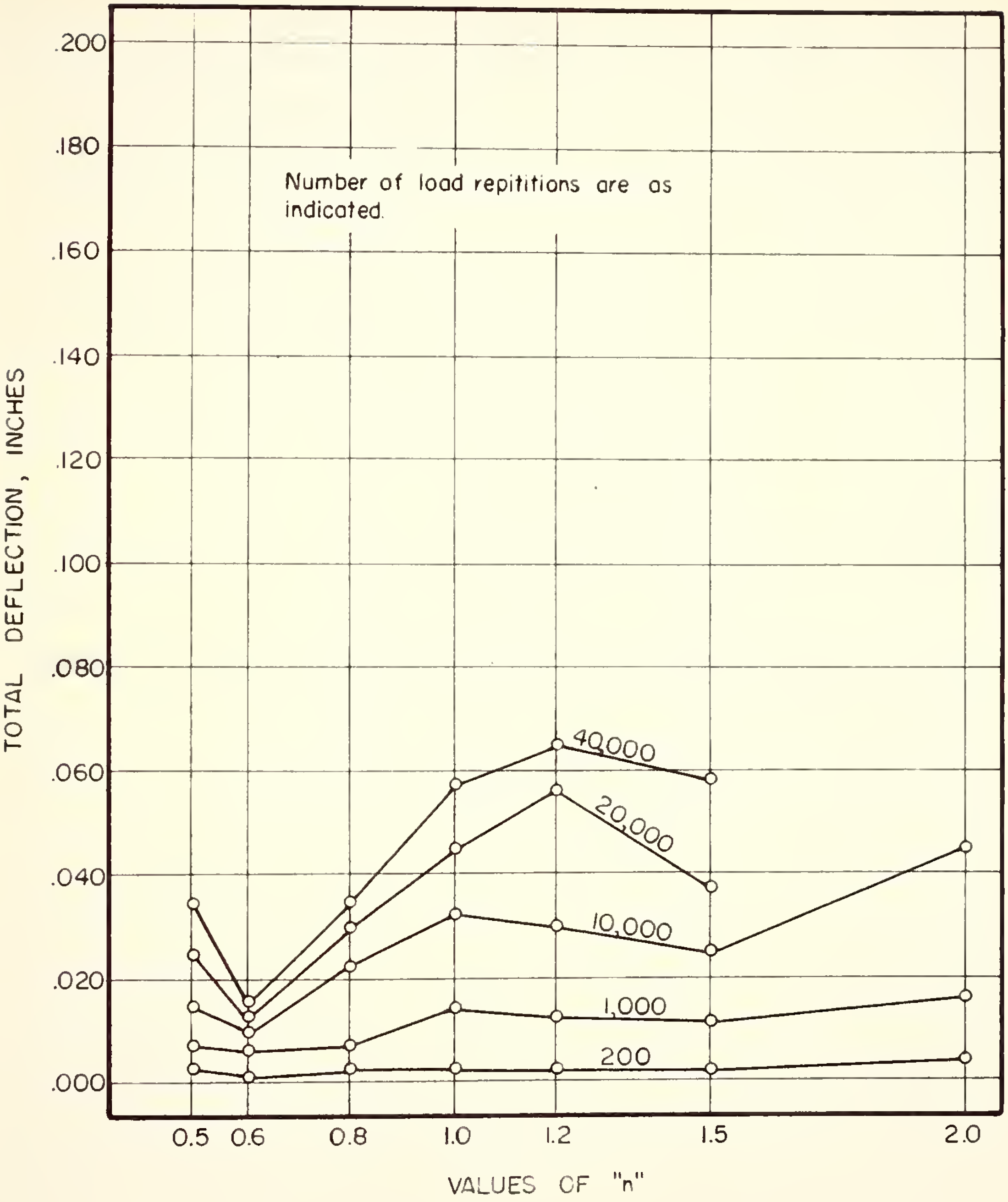


FIG. 19A* DEFLECTION VERSUS GRADATION FOR SAMPLES WITH SAND BASE COURSE

* Figure added after completion of report.

less increase in minus No. 200 material in the base than did the less open-graded sample 3/4 - 1.5. This could probably be attributed in small part to chance differences in subgrade sample preparation (table 10) but was felt to be due mainly to the great difficulty in obtaining uniformity in the base course and preventing segregation during placing which was particularly evident with the open-graded gravels.

In only one case (4 - 0.5) did a test specimen with a sand base course exhibit pumping of fine soil from the base, and this was very minor in extent (see Table 8). In three cases (4 - 1.2, 4 - 1.5, and 4 - 2.0) a small amount of subgrade intrusion was noted. This also was shown to be minor in extent by the results of a grain size analysis performed on the base course sample after testing (Table 8). In general, specimens with sand base courses showed less material pumped to the base course surface, less intrusion of subgrade soil and small deflections over a wider range of gradation than those with gravel base courses, even though greater percentages of minus No. 200 soil existed in the sand samples than in corresponding gravels.

These results probably reflect the smaller pore sizes which would exist in the finer of two similarly graded materials with different maximum aggregate sizes. Also, greater elastic deflections were generally observed in specimens with gravel base courses.

The three base course samples (3/4 - 0.4, 4 - 0.5 and 4 - 0.6) which meet the filter requirements adopted by the Corps of Engineers (10) are all dense-graded materials which appeared to allow no significant intrusion of subgrade soil during the repeated load tests. From the results of this series of tests, it appears that the criteria may be

adequate for the prevention of subgrade intrusion and, in fact, somewhat conservative since it excludes most of those samples in which no significant intrusion occurred. More important, perhaps, is the fact that most materials that would meet the requirements of this criteria for the particular soil used in this study would probably be susceptible to the pumping of material to the base course surface if subjected to the conditions imposed on the specimens tested in this investigation.

It can be said that a gravel base course sample graded anywhere between the limits expressed by the formulas $p = 100(\frac{d}{D})^{0.7}$ and $p = 100(\frac{d}{D})^{1.2}$ would probably perform satisfactorily (no excessive deflections due to subgrade intrusion or base course pumping) if subjected to the condition of this repeated loading test. In practice, these limits might be extended to include a wider range of gradation since less severe conditions of load would be anticipated. All else being equal, base courses constructed with a coarse sand could be expected to perform satisfactorily over a wider range of gradation than those constructed with a larger sized material.

CONCLUSIONS

Based solely on the results of this laboratory investigation and the experience gained in conducting it, the following conclusions appear to be justified:

1. Under the proper conditions of moisture and load, a dense-graded base course material with an appreciable percentage of material finer than the No. 200 mesh sieve could be expected to have some of its finer soil sizes removed by a pumping action; and a very open-graded base course material could be expected to be contaminated by the intrusion of subgrade soil. In the series of tests here reported, movement of soil to the base course surface occurred where an excess of 3 per cent by weight of soil finer than the No. 200 mesh sieve was present in the gravel bases and where an excess of 12 per cent was present in the sands.

2. Since a good correlation was found to exist between large subgrade-base course deflections and either the intrusion of subgrade soil into the base course or the removal of fine soil from the base course by a pumping action, it might be concluded that an appreciable degree of subgrade intrusion or base course pumping will result in objectionable deflections of the subgrade-base course system.

3. The principle effect of a decrease in the degree of base course compaction appeared to be a greater initial settlement within the early service life of the subgrade-base course system, presumedly, primarily within the base course.

4. An optimum range of gradation appears to exist within which a base course sample may experience neither significant subgrade intrusion nor movement of fine soil to the base course surface. For the series of laboratory tests on specimens with gravel base course, this range existed approximately between the grain size distribution curves expressed by the formulas $p = 100(\frac{d}{D})^{0.7}$ and $p = 100(\frac{d}{D})^{1.2}$. In practice, this range could be extended in anticipation of less severe conditions of loading.

5. All else being equal, base courses consturcted with a coarse sand could be expected to perform satisfactorily over a wider range of gradation than those with a larger maximum particle size.

6. If a base course material is graded within this optimum range and compacted to a high relative density over a well prepared subgrade, deflections of the subgrade-base course system should not exceed that amount generally considered as tolerable for rigid pavements (around 0.05 inch).

7. With respect to the density tests performed on each of the fifteen base course gradations:

a. The unit weights that can be attained with a granular material are a function of the gradation of that material with a maximum value attainable when the aggregate is graded to some approximation of Fuller's theoretical curve of maximum density.

b. There is a certain range of gradation about Fuller's curve where small changes in the grain size distribution of an aggregate markedly effect the unit weights that can be attained.

c. There is a significant amount of aggregate breakage in the

dynamic compaction test according to CE Specification 807.1 (13), such that, in some cases, unit weights may be indicated for a given material that exceed the maximum unit weight that can be attained for that material when its gradation is not altered by breakage.

8. It appears that the filter criteria adopted by the Corps of Engineers, U. S. Army (10) for the protection of thin base courses against the intrusion of subgrade soil are adequate and, in fact, may be somewhat conservative, insofar as intrusion resulting from a kneading action is concerned.

9. The results of this investigation indicate that more consideration might justifiably be given to sand as a more satisfactory base course material than gravel.

BIBLIOGRAPHY

1. Allen, C.W. and Childs, L.D., "Report on Pavement Research Project in Ohio", Highway Research Board, *Bulletin 116*, 1956.

2. American Association of State Highway Officials, *Standard Specifications for Highway Materials and Methods of Sampling and Testing*, Part II, Washington, 1955.

3. Belcher, D.J., Gregg, L.E., and Woods, K.B., "The Formation, Distribution and Engineering Characteristics of Soils," *Highway Research Bulletin No. 10*, Purdue University, Lafayette, Indiana, January, 1943.

4. Bertram, G.E., "An Experimental Investigation of Protective Filters," *Soil Mechanics Series No. 7*, Graduate School of Engineering, Harvard University, January, 1940.

5. Buchanan, S.J. and Khuri, F.I., "Elastic and Plastic Properties of Soils and their Influence on the Continuous Support of Rigid Pavements," *Research Report* prepared for the Corps of Engineers, U.S. Army, June, 1954.

6. Burmister, D.M., "The Grading-Density Relations of Granular Materials", *Proceedings*, American Society for Testing Materials, Vol. 38 1938.

7. Burmister, D.M., "The Importance and Practical Use of Relative Density in Soil Mechanics", *Proceedings*, American Society for Testing Materials, Vol. 48, 1949.

8. Burmister, D.M., "A Study of the Physical Characteristics of Soils with Special Reference to Earth Structures", *Bulletin No. 6* Civil Engineering Research Laboratories, Columbia University, June, 1938.

9. Corps of Engineers, U.S. Army, "Airfield Pavement Design, Rigid Pavements", *Engineering Manual for Military Construction*, Part XII, Chapter 3, May, 1956.

10. Corps of Engineers, U.S. Army, "Airfield Pavement Design, Frost Conditions", *Engineering Manual for Military Construction*, Part XII, Chapter 4, October, 1954.

11. Corps of Engineers, U.S. Army, "Drainage and Erosion Control -- Subsurface Drainage Facilities for Airfields", *Engineering Manual for Military Construction*, Part XIII, Chapter 2, July, 1946.

12. Corps of Engineers, U.S. Army, "Investigation of Filter Requirements for Underdrains", *Technical Memorandum No. 183-1*, U.S. Waterways Experiment Station, Vicksburg, Mississippi, November, 1941.

13. Corps of Engineers, U.S. Army, *Pavement, Flexible, Select Material Base Course, Spec. CE 807.1*, March, 1946, revised, 1952.

14. Corps of Engineers, U.S. Army, "Rigid Pavement Pumping Experience Frost Investigations, 1953", Arctic Construction and Frost Effects Laboratory, New England Division, 1954.

15. Corps of Engineers, U.S. Army, "The Unified Soil Classification System", *Technical Memorandum No. 3-357*, Waterways Experiment Station, Vicksburg, Mississippi, March, 1953.

16. Fuller, W.B. and Thompson, S.E., "The Laws of Proportioning Concrete", *Transactions*, American Society of Civil Engineers, Vol. LIX, 1907.

17. Havers, J.A., "A Study of the Interactions of Selected Combinations of Subgrade and Base Course Subjected to Repeated Loading", *A paper*, presented at the Thirty-Sixth Annual Meeting of the Highway Research Board, Washington, D.C., January, 1957.

18. Henderson, E.A. and Spencer, W.T., "Effect of Experimental Subgrade Treatments on Pavement Faulting and Pumping", *Proceedings*, Highway Research Board, Vol. 26, 1946.

19. Highway Research Board, "Final Report on Road Test One - Md.", *Special Report 4*, National Academy of Science, National Research Council, Publication 227, 1952.

20. Indiana, State Highway Commission, *Standard Specifications for Road* and Bridge Construction and Maintenance, 1952.

21. Indiana, State Highway Commission, Various unpublished records of construction.

22. Karpoff, K.P. "The Use of Laboratory Tests to Develop Design Criteria for Protective Filters", *Proceedings*, American Society for Testing Materials, Vol. 55, 1955.

23. Keene, Philip, "Underdrain Practice of the Connecticut Highway Department", Proceedings, Highway Research Board, Vol. 24, 1944.

24. Lambe, T.W., *Soil Testing for Engineers*, John Wiley & Sons, Inc., New York, 1951.

25. Lane, Kenneth S., "Providence Vibrated Density Test", Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, Vol. IV, 1948.

26. Leonards, G.A., "Strength Characteristics of Compacted Clays", Transactions, American Society of Civil Engineers, Vol. 120, 1955.

27. McLaughlin, J.F., "The Load-Carrying Characteristics of a Bituminous Concrete Resurfacing Mixture", A Thesis submitted to the faculty of Purdue University in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy, Purdue University, January, 1957.

28. Shelburne, T.E., "Crushing Resistance of Surface-Treatment Aggregation", Research Series No. 76, Highway Research Bulletin No. 5, Purdue University, Lafayette, Indiana, September, 1940.

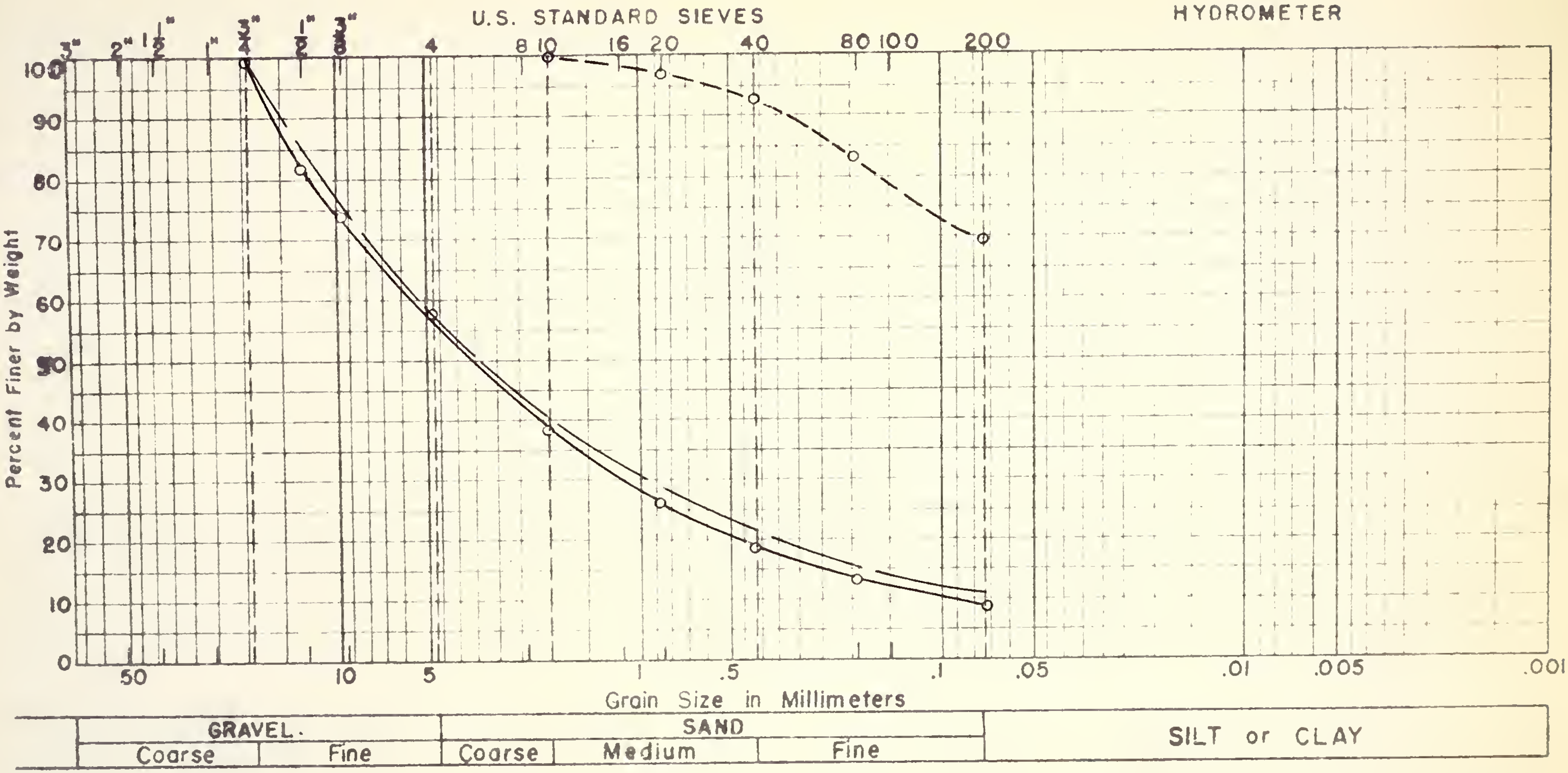
29. Spencer, W.T., Allen, H., and Smith, P.C., "Report on Pavement Research Project in Indiana", Highway Research Board, Bulletin 116, 1956.

30. Talbot, A.N. and Richart, F.E., "The Strength of Concrete; Its Relation to the Cement Aggregate and Water", Bulletin 137, Engineering Experiment Station, University of Illinois, October 15, 1923.

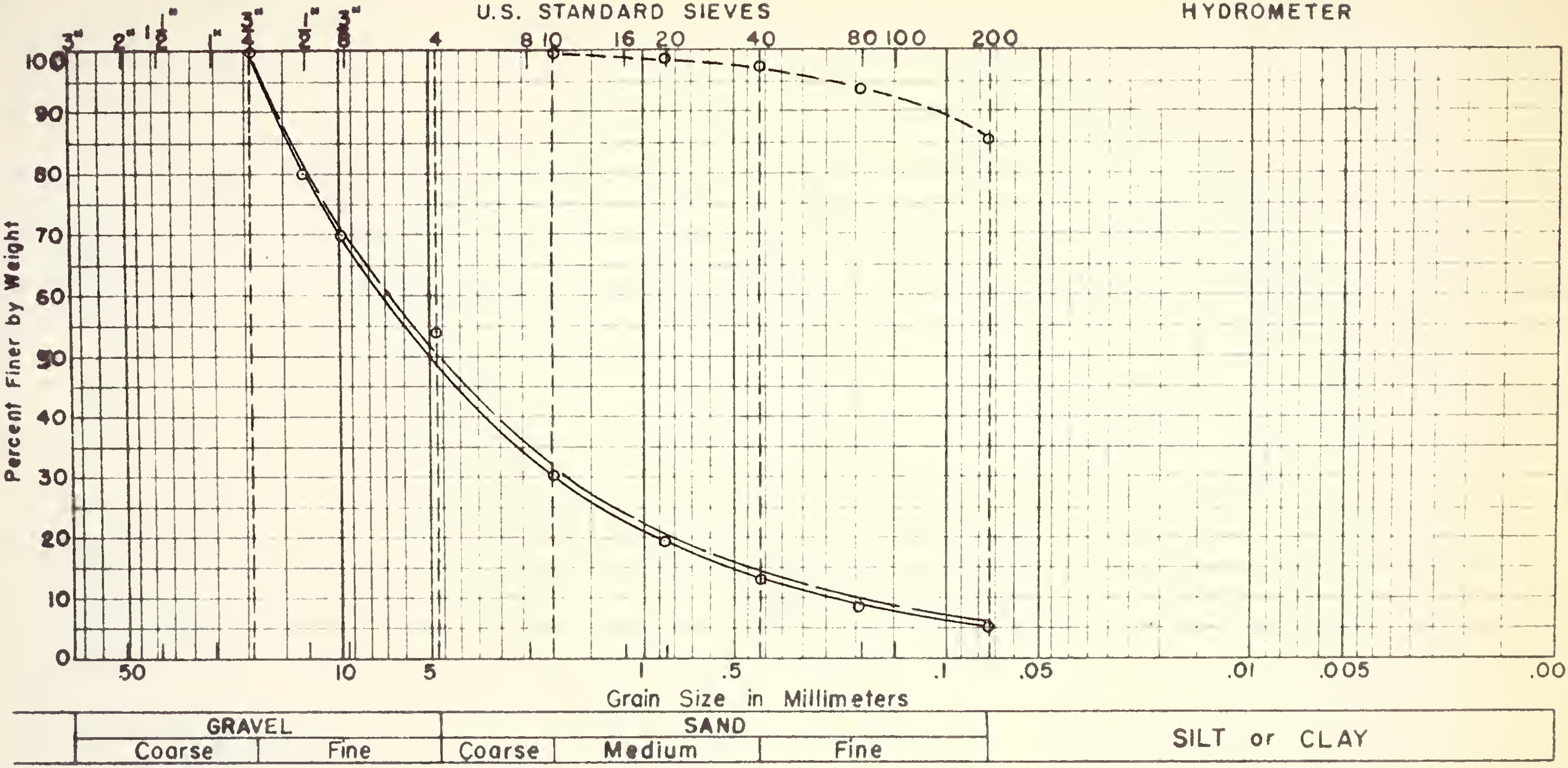
31. Terzaghi, Karl and Peck, R.B., Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1948.

32. Yoder, E.J., "Pumping of Highway and Airfield Pavements", A paper presented at the Thirty-Sixth Annual Meeting of the Highway Research Board, Washington, D.C., January, 1957.

APPENDIX



GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST.
SAMPLE 3/4 -0.4
FIG. 20



Remarks

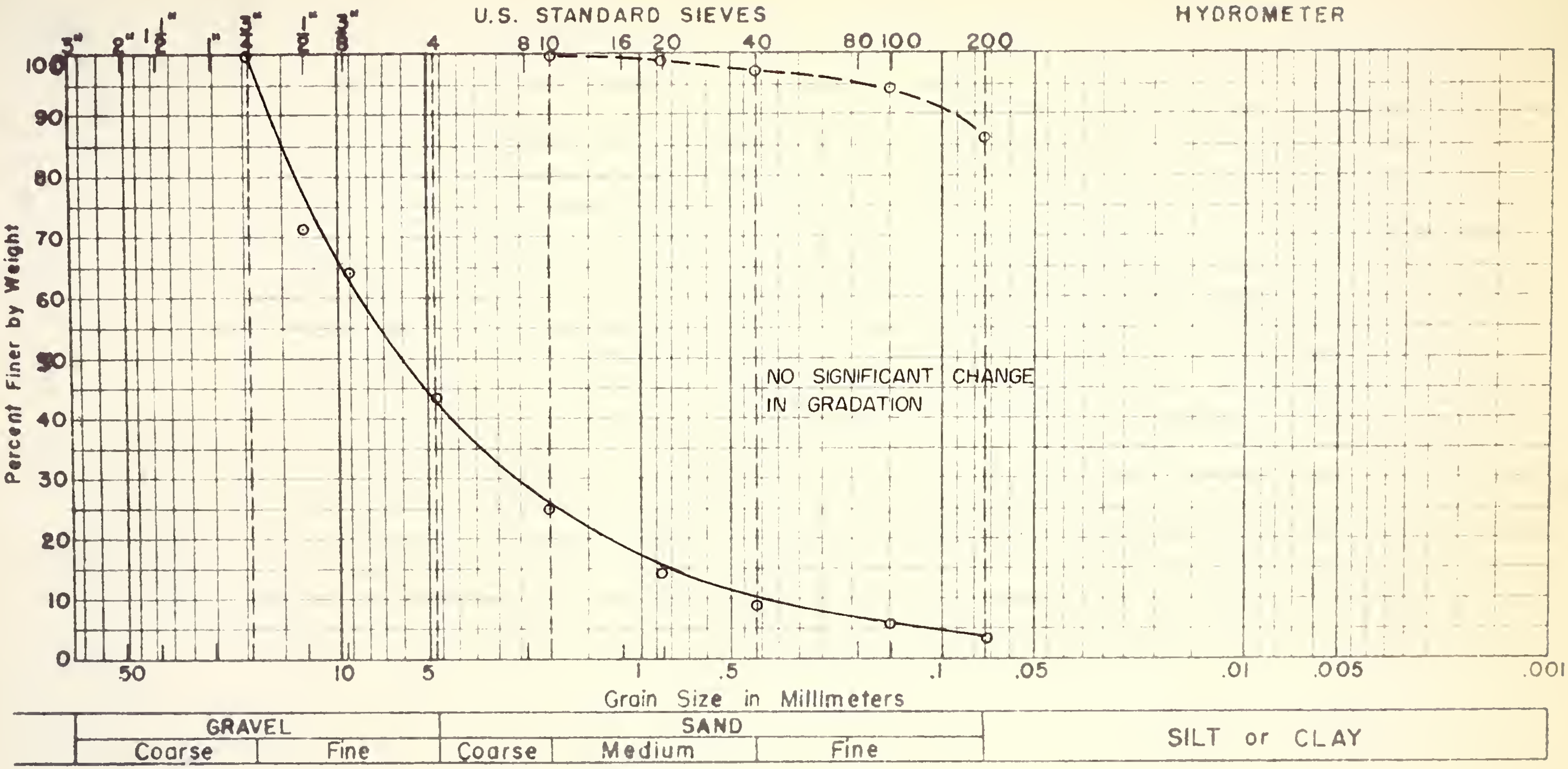
LEGEND

——— ——— BASE BEFORE TEST

o————o BASE AFTER TEST

o- - - - - -o PUMPED MATERIAL

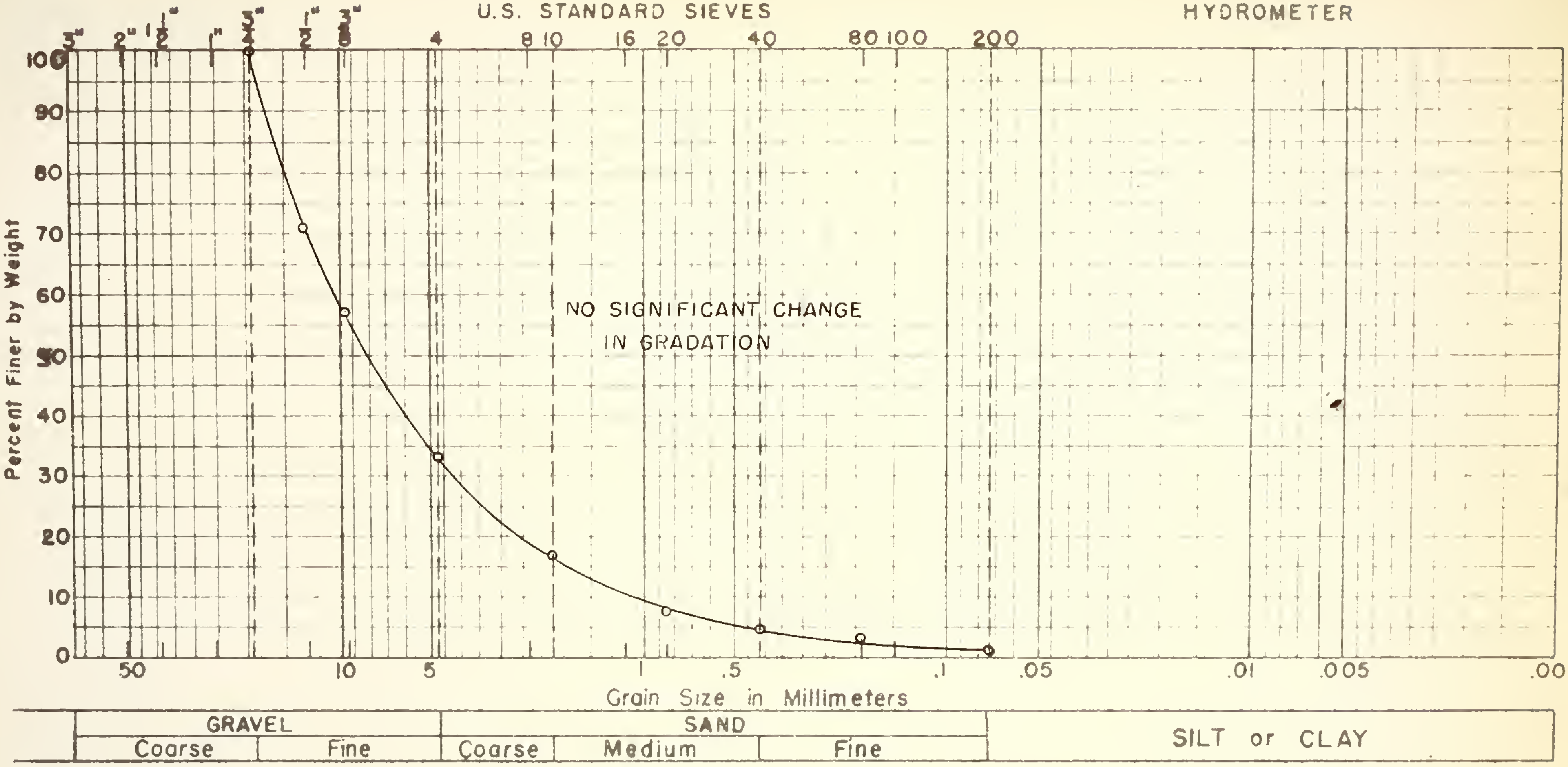
GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE ¾ -0.5
FIG. 21



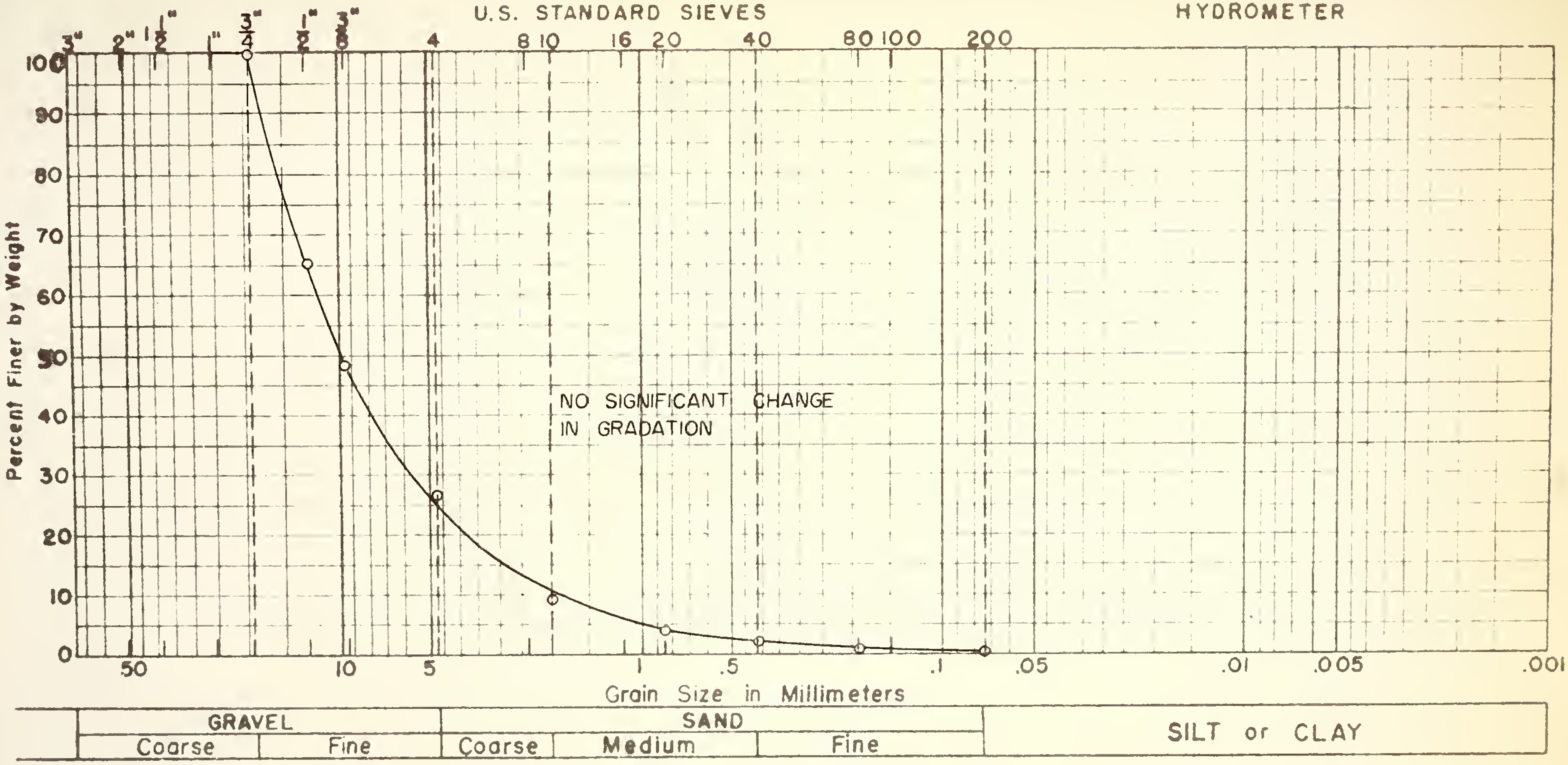
Remarks

LEGEND
- BASE BEFORE TEST
- BASE AFTER TEST
- PUMPED MATERIAL

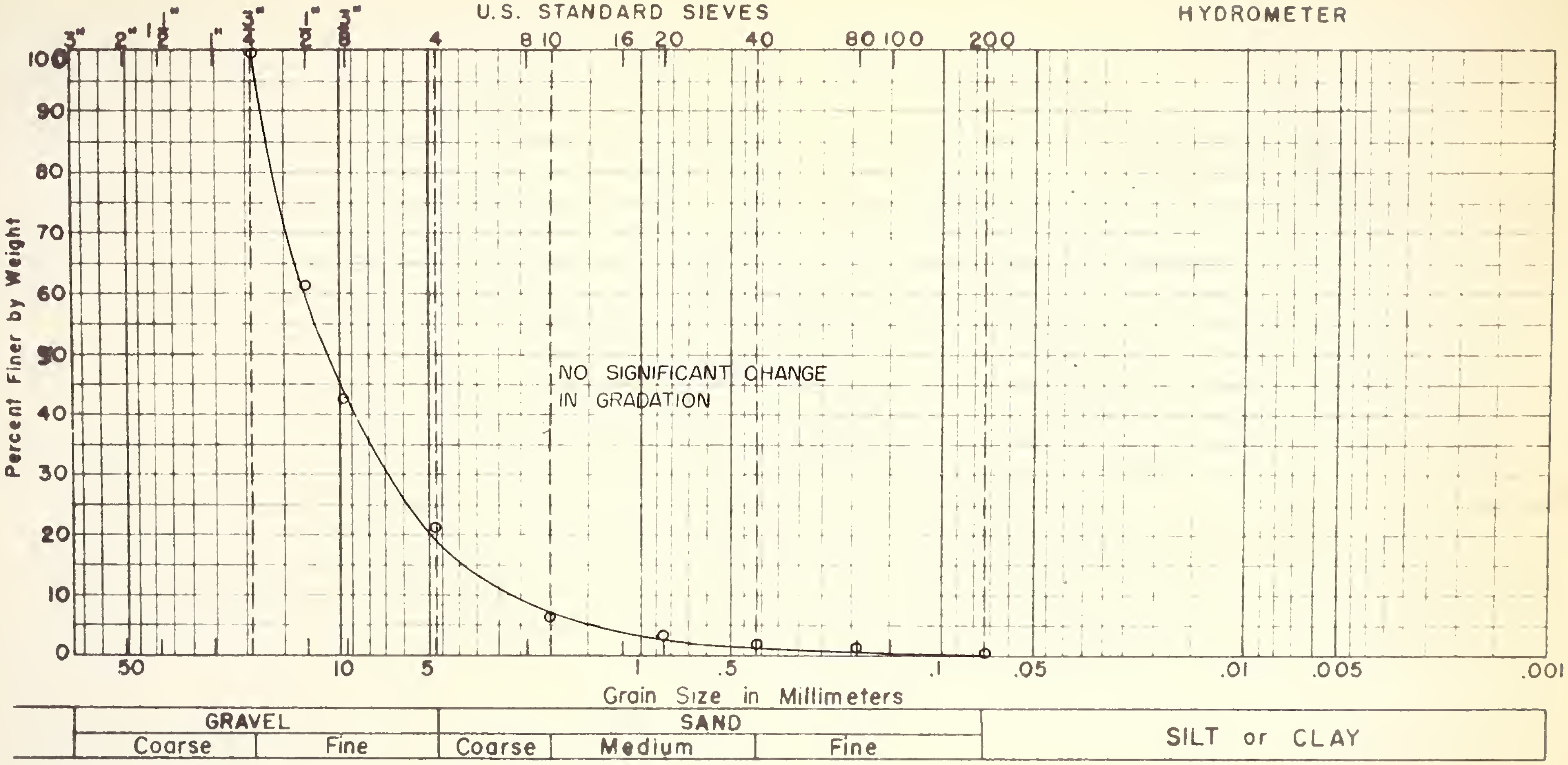
GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE ¾ -0.6
FIG. 22



GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE ¾ -0.8
FIG. 23



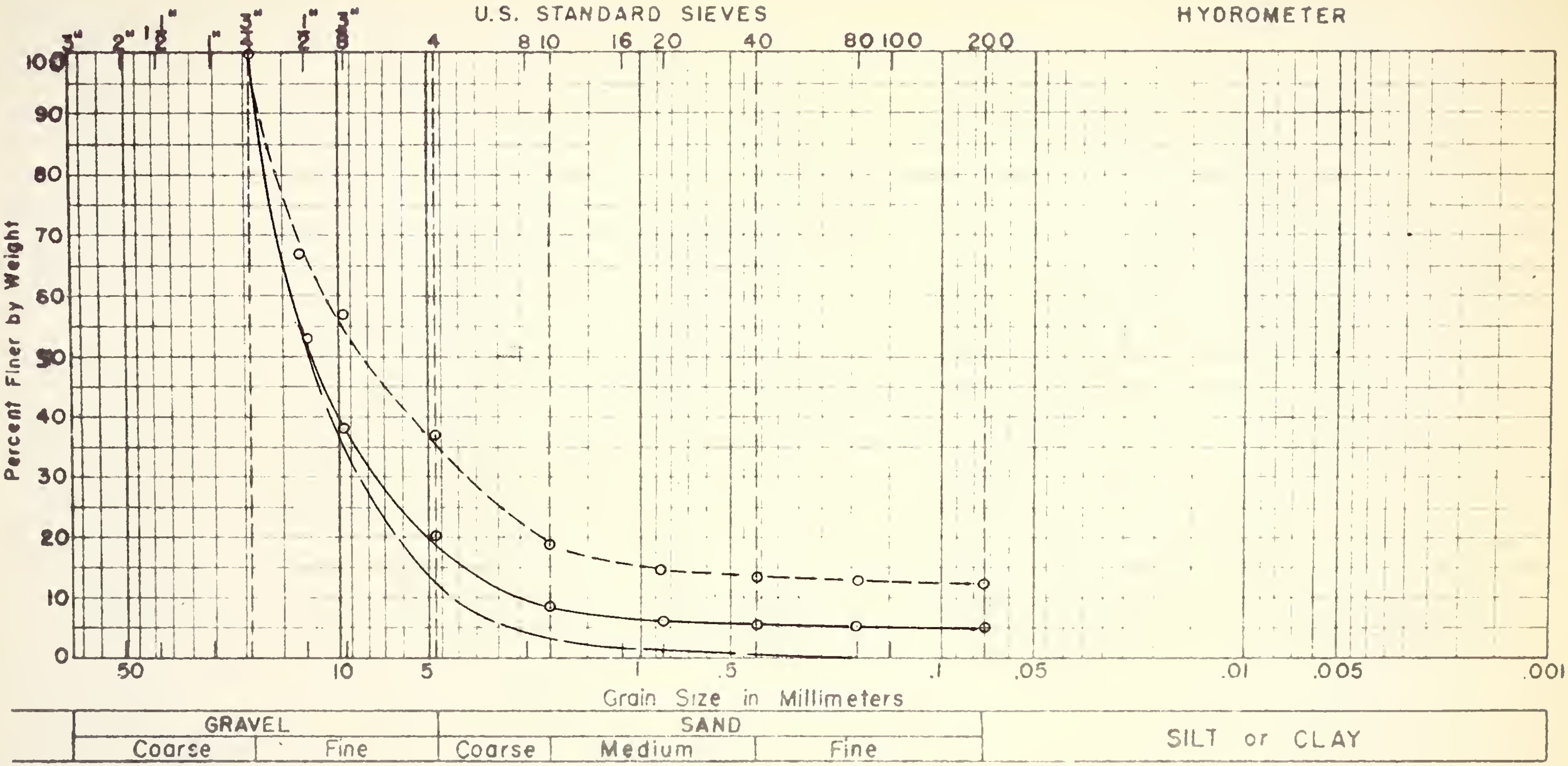
GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE ¾-1.0
FIG. 24



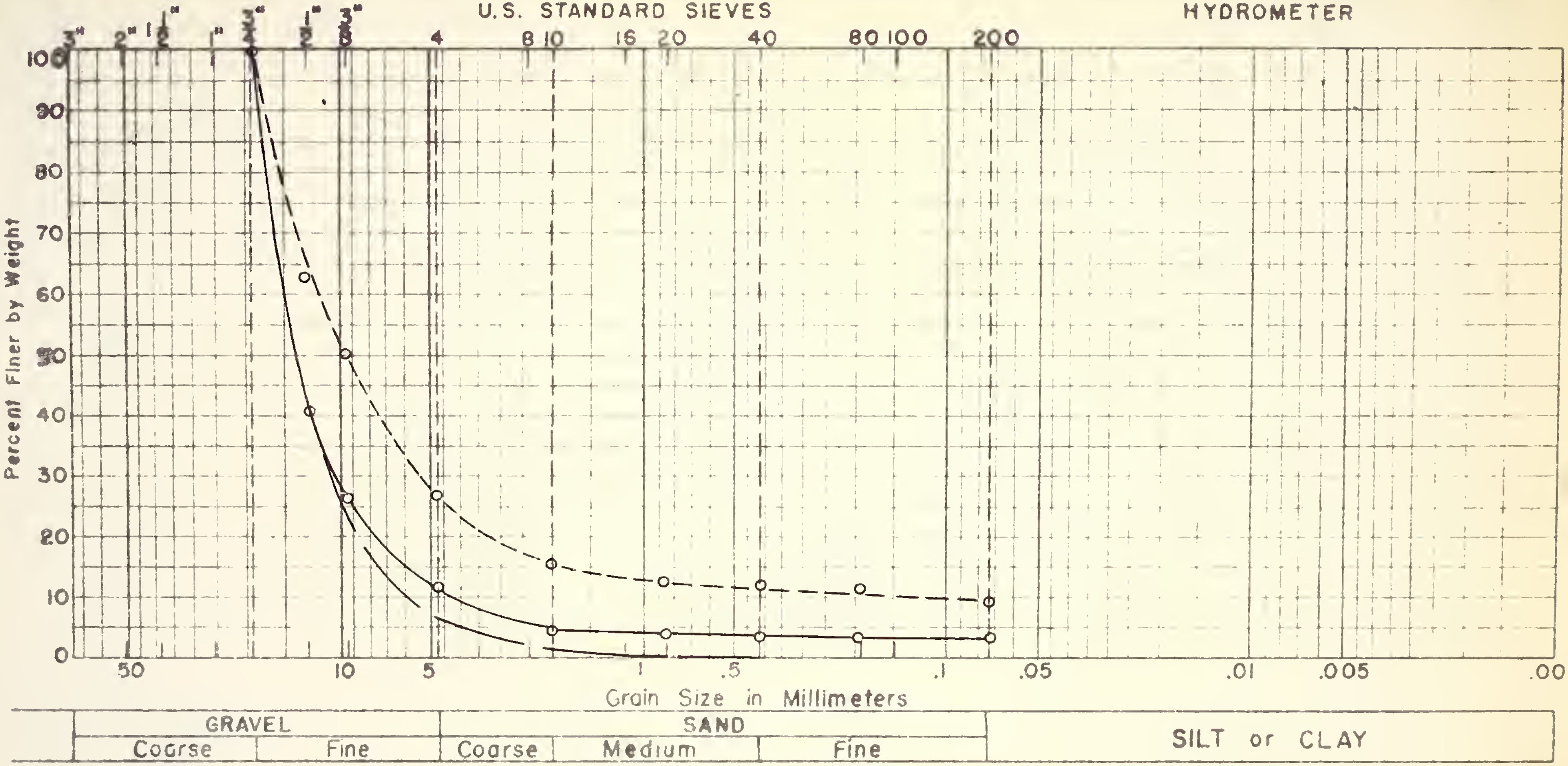
Remarks

LEGEND
— — BASE BEFORE TEST
○—○ BASE AFTER TEST

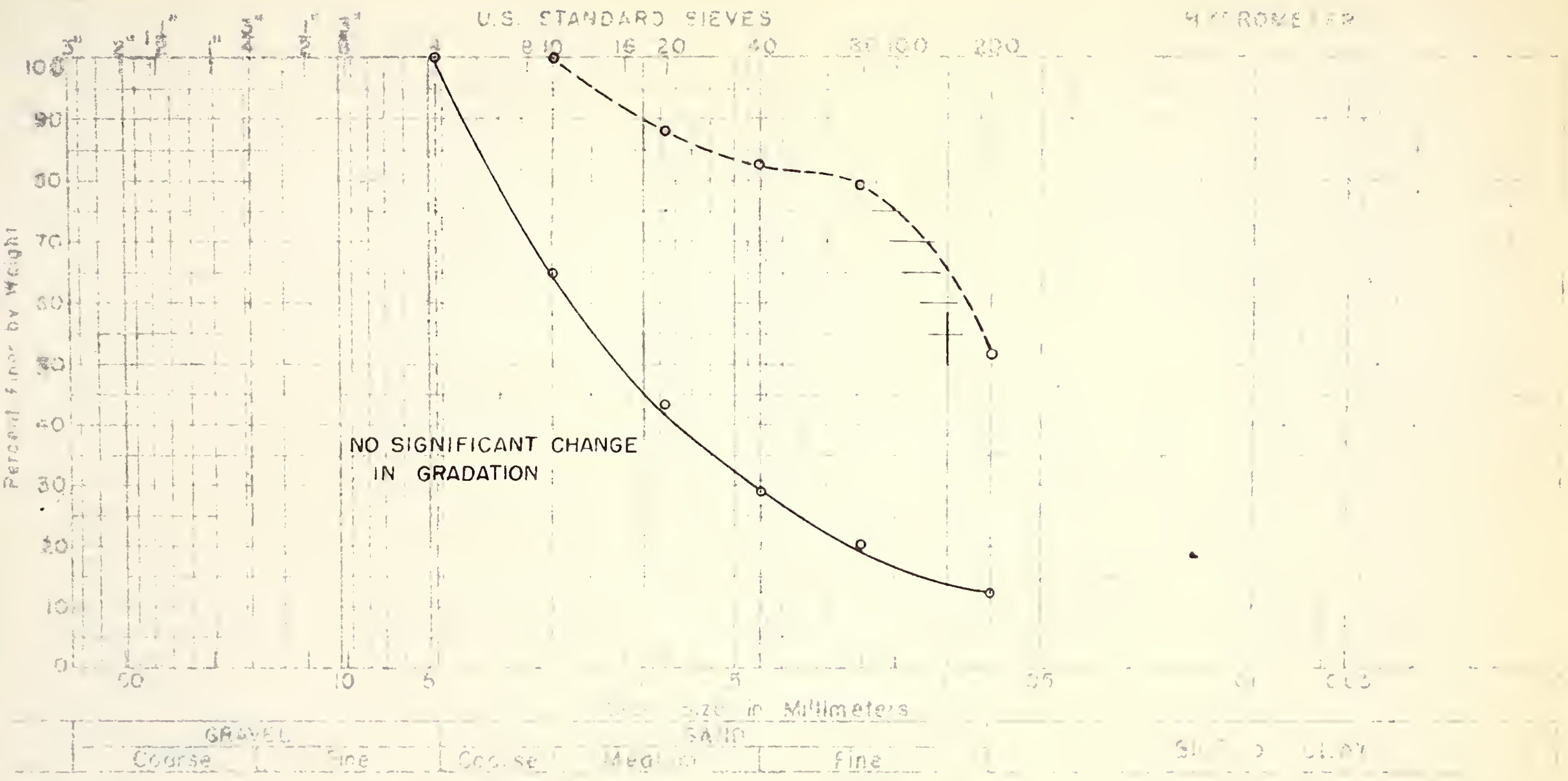
GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE ¾-1.2
FIG. 25



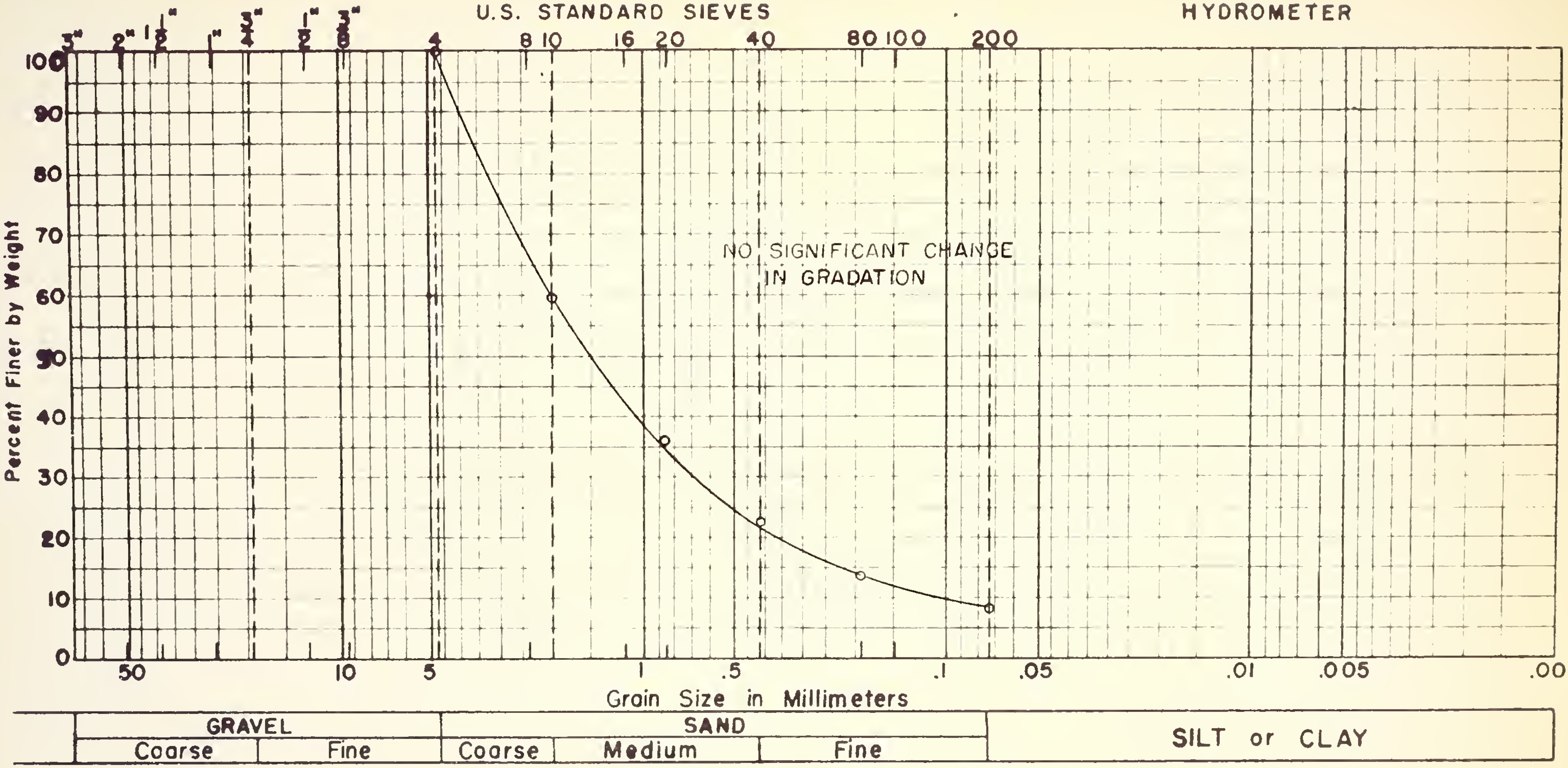
GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 3/4 -1.5
FIG. 26



GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 3/4-2.0
FIG. 27



GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4-0.5
FIG. 28



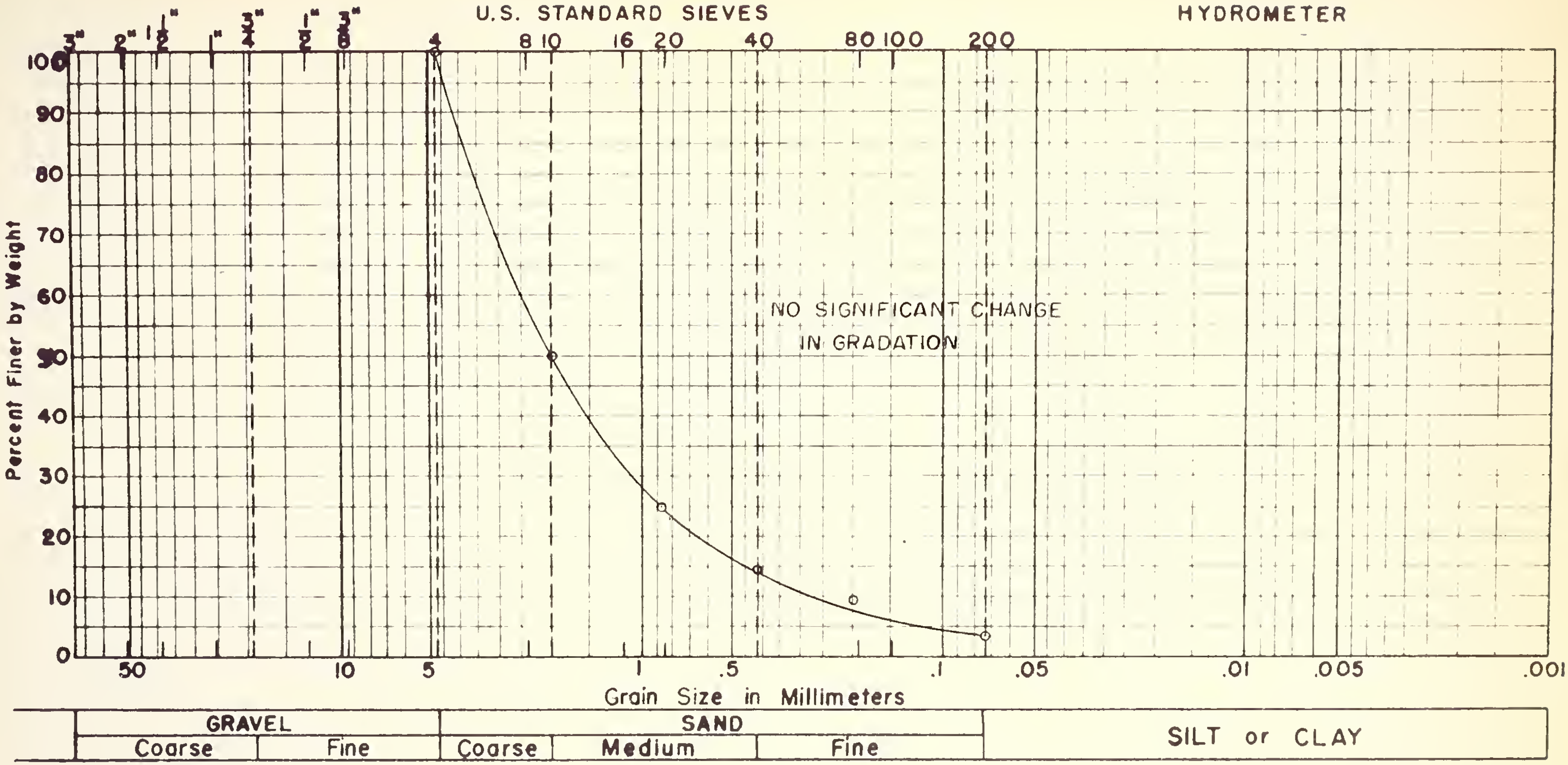
Remarks

LEGEND

—— —— BASE BEFORE TEST

o——o BASE AFTER TEST

GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4-0.6
FIG. 29



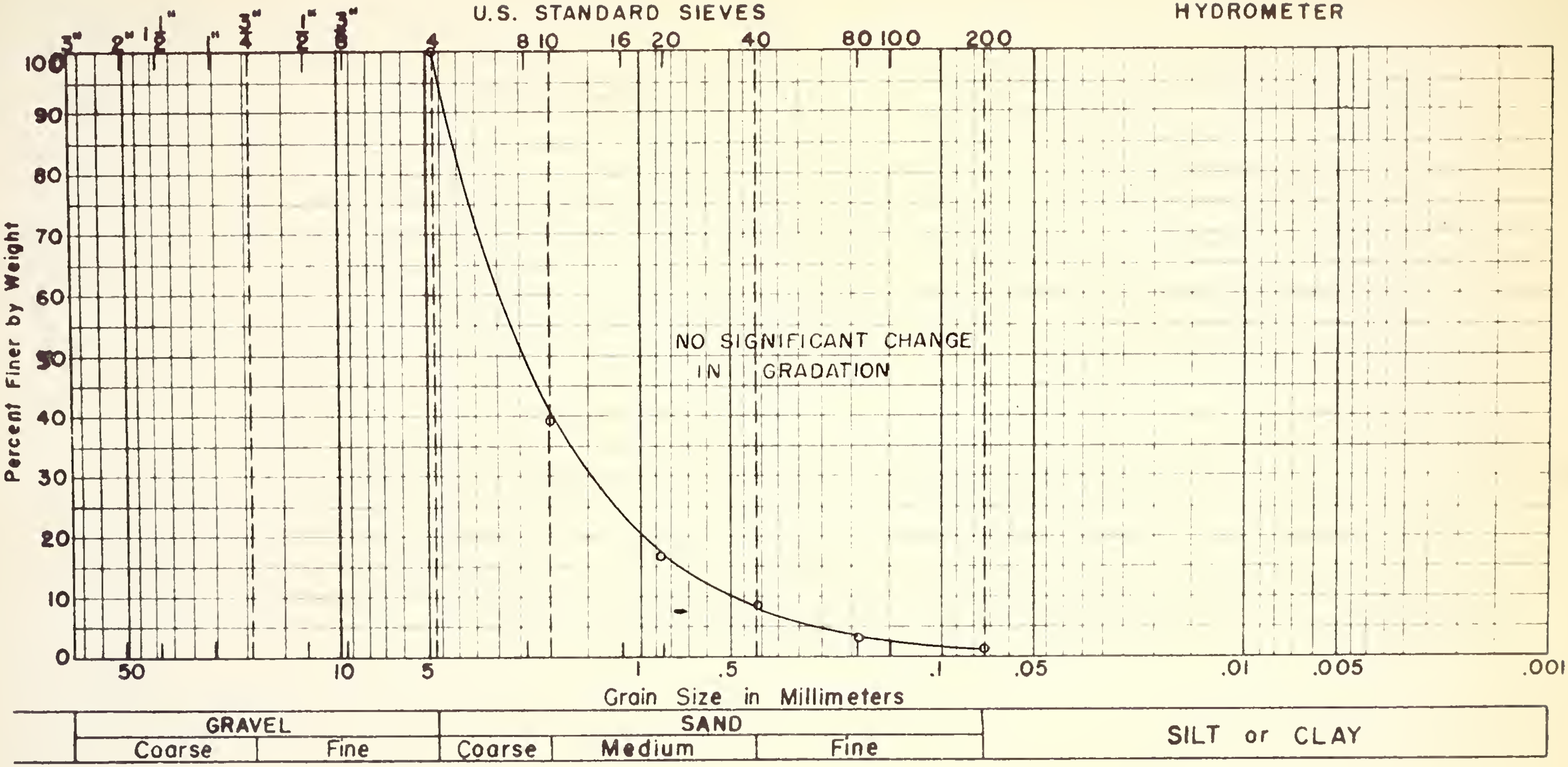
Remarks

LEGEND

——— ——— BASE BEFORE TEST

o————o BASE AFTER TEST

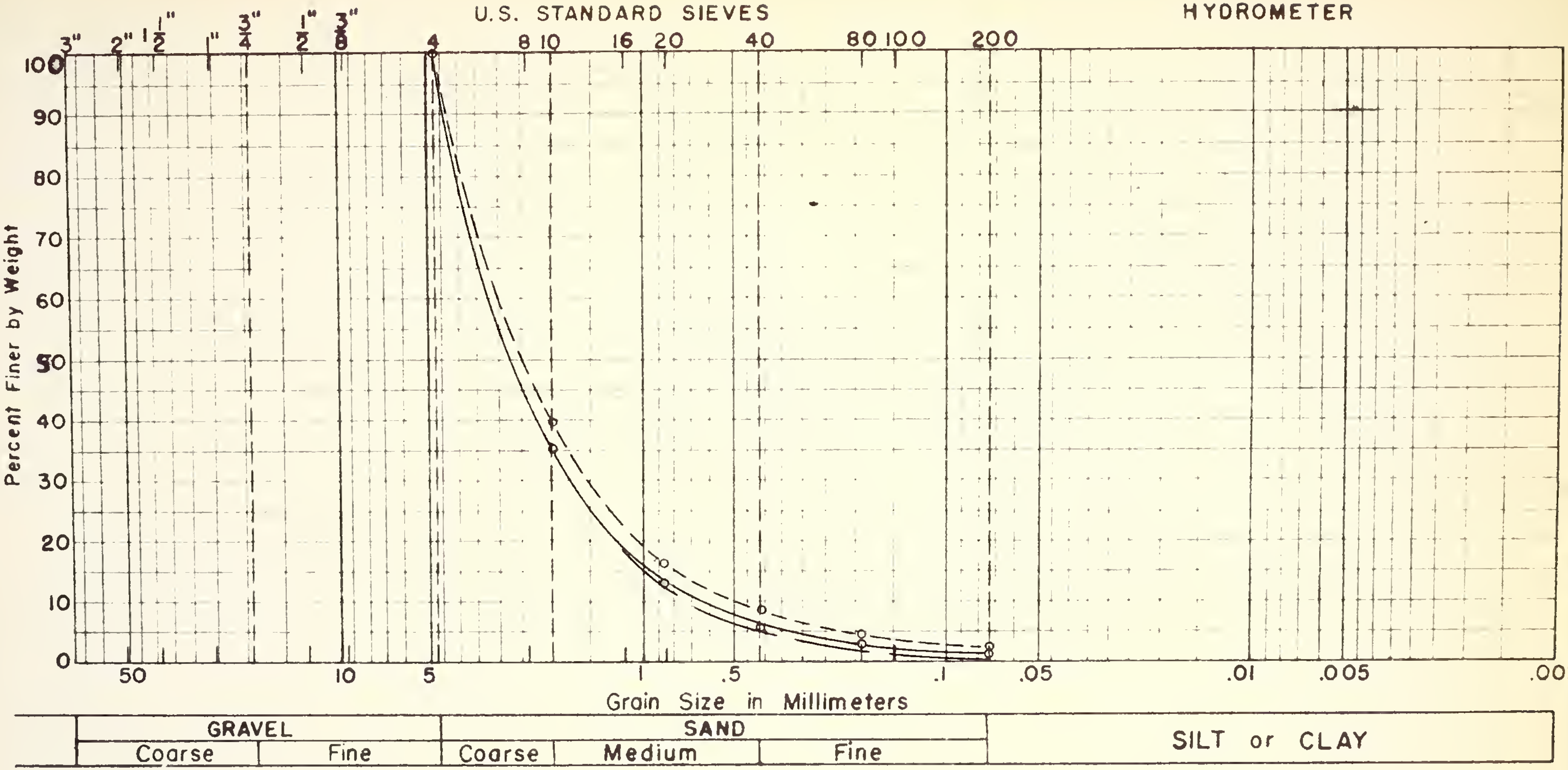
GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4-08
FIG. 30



Remarks

LEGEND
BASE BEFORE TEST
○——○ BASE AFTER TEST

GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4-10
FIG. 31



Remarks

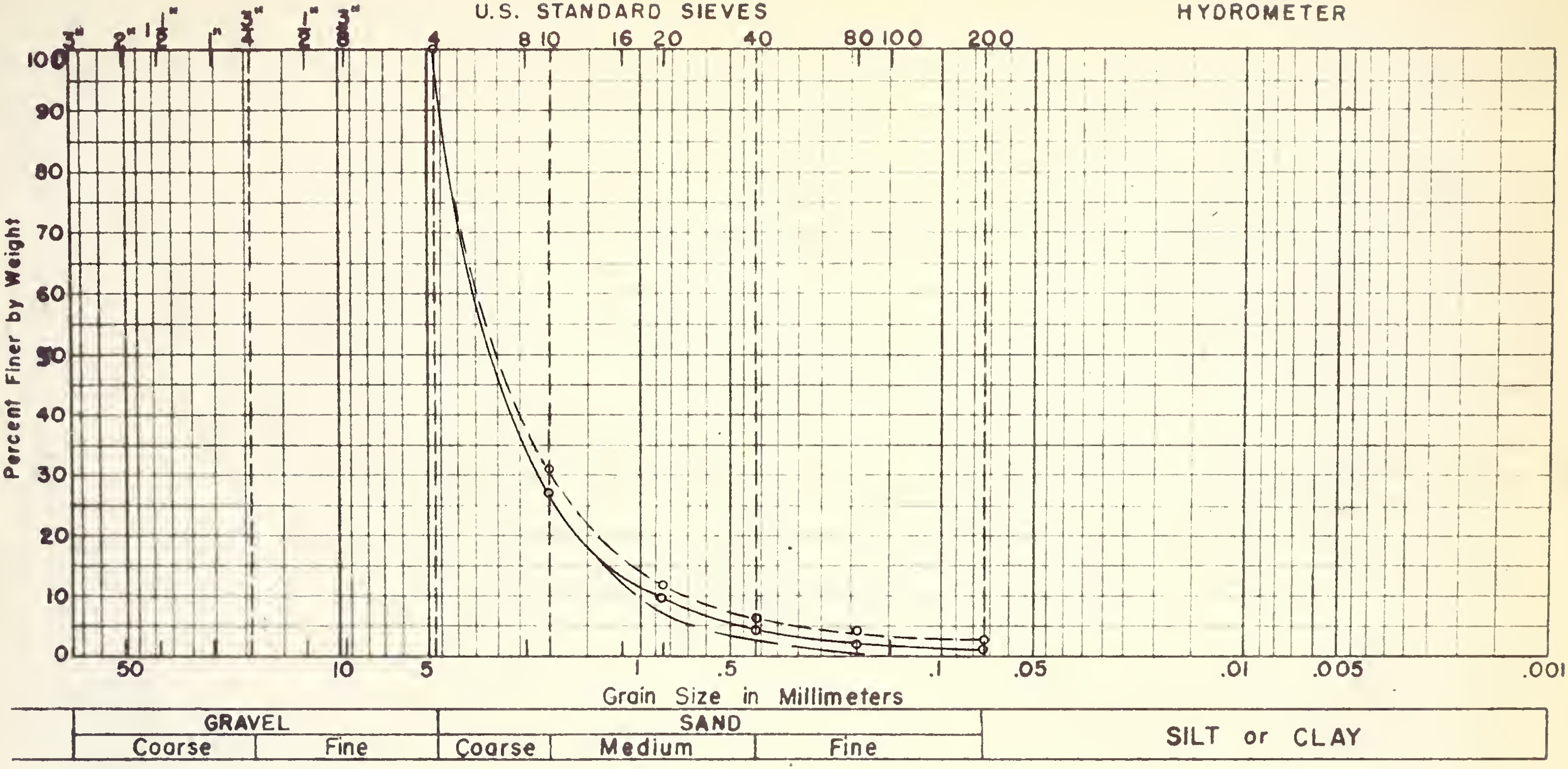
LEGEND

——— ——— BASE BEFORE TEST

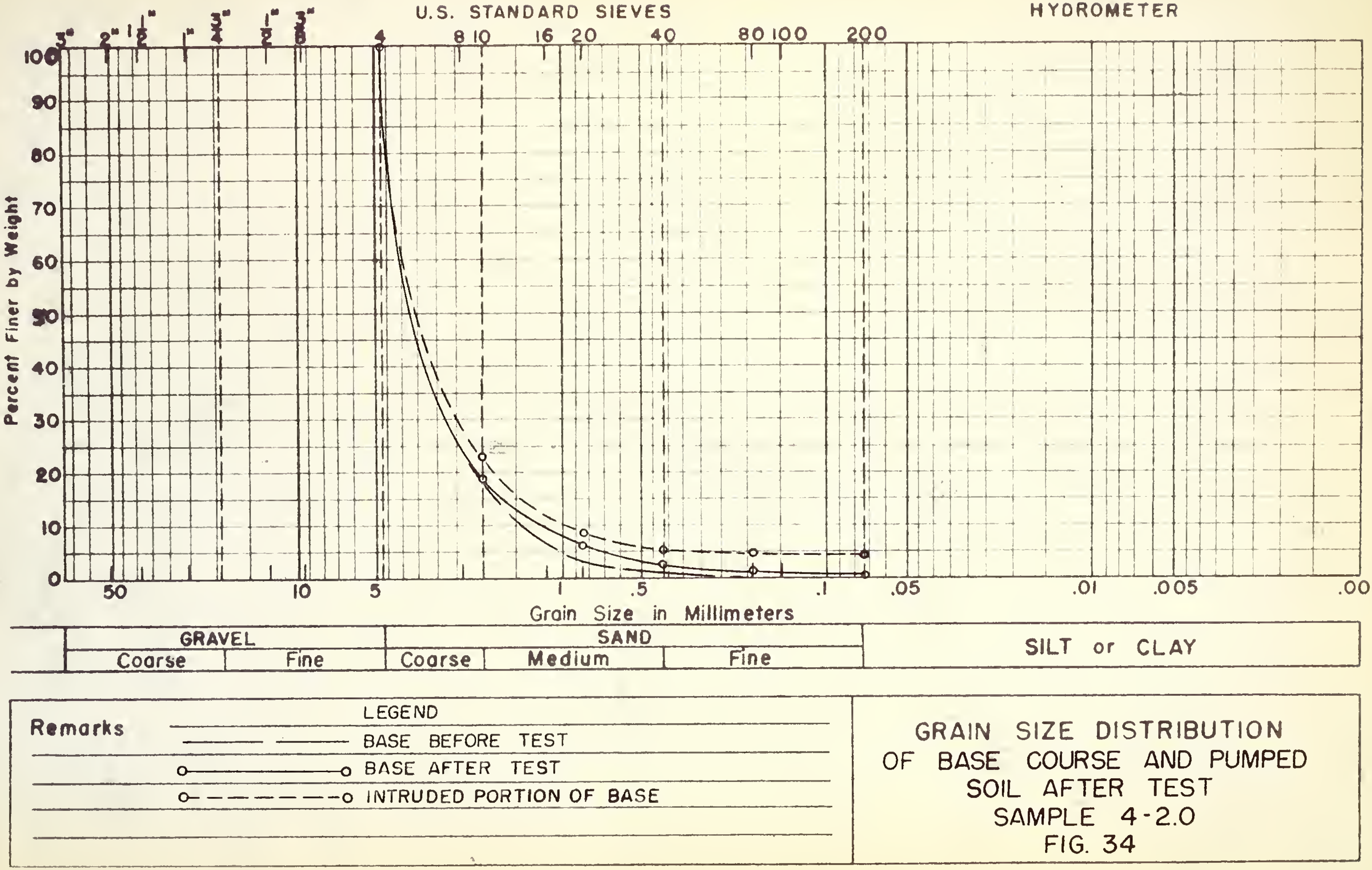
o————o BASE AFTER TEST

o- - - - - -o INTRUDED PORTION OF BASE

GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4-1.2
FIG. 32



GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4 -1.5
FIG. 33



GRAIN SIZE DISTRIBUTION
OF BASE COURSE AND PUMPED
SOIL AFTER TEST
SAMPLE 4-2.0
FIG. 34